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DAMAGE FUNCTIONS FOR UPGRADED SHELTERS. (U)
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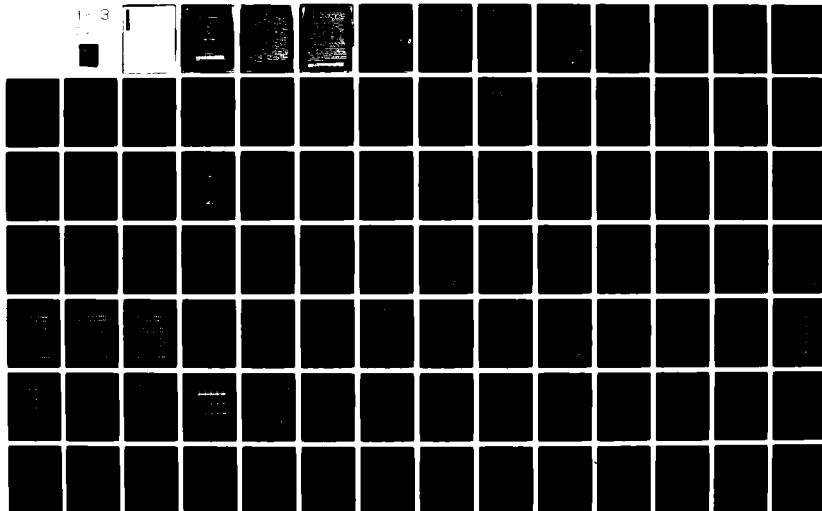
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FEMA Contract EMB-C-0074

FINAL REPORT
SUMMARY

By

A. Longino
M.T. W.
J. Schomall

for

Federal Emergency Management Agency
Washington, D.C. 20472

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SUMMARY

At some future time existing buildings in this country may need to be surveyed and identified for expedient upgrading to provide fallout protection and blast protection for evacuees and workers of critical importance in the event of a nuclear weapon attack. Expedient upgrading refers to methods that may be employed to strengthen basements of existing buildings using readily available materials and skilled or semiskilled labor during a short period.

The study described here was concerned with predicting the probability of survival of people located in expediently upgraded, conventional basements when subjected to the blast effects produced by the detonation of a 1000-ton weapon near the ground surface. Two categories of potential shelters are considered here, i.e., engineered buildings and framed single family residences. These are described.

The first category refers to low rise engineered buildings with basements. The basement walls, both exterior and interior are unexposed and the first floor slab, the slab over the basement, is at grade. The first floor slab is thus the primary structural component for the basement as far as protection from blast is concerned. Its collapse will result in cascading debris from the collapsed slab and due to blast pressures and blast waves entering shelter areas when the shelter envelope is breached.

The slab over the basement was designed as a one-way slab subject to five loads of 50, 80, 125, and 250 psf and span lengths of 10, 15, and 20 ft. This includes a total of twelve conventional basements representing a wide range of building use classes. Each of the basements was upgraded in this study as expediently upgraded using low resistance materials. Upgrading is accomplished by providing support for the first floor slab. The effective span is reduced, and by shearing off all exterior walls. This resulted in sixty shelters of different strengths and sizes. The conventional basements were upgraded as follows:

The second category refers to four conventional wood-frame family dwellings with full basements. Detailed upgrading schemes in this portion of the study include the stepped and the post and beam concepts. They were used to reduce by half the effective span of the floor joists. In addition to strengthening the floor system, upgrading also implies blocking off all openings into the basement and surrounding the building on the outside with soil up to the first floor level. For the buildings considered here this amounts to erect a vertical barrier of 2 to 3 ft.

Six shelters were evaluated. First, each basement was upgraded and regraded using the stepped concept; second, two of the basements were re-evaluated using the post and beam concept. The process was repeated by assuming that 1 ft of soil would be placed over the first floor for radiation protection. Placing 2 ft of soil would significantly affect the strength of the floor system of these basements. The case involving 2 ft of soil for radiation protection was, therefore, not considered.

A probability of survival function was developed for each shelter and each upgrading scheme. The procedure used to accomplish this consists of two parts. The first is a probabilistic structural analysis which determines the probability of failure to the shelter envelope. The second is a probabilistic people survival analysis which considers two casualty producing mechanisms, i.e., debris impact and associated effects from the collapse of the basement slab, and primary blast. The probability of structural failure is then used in computing the probability of survival against debris impact.

The analysis method briefly described above was developed in the course of the previous effort and this study. A portion of this method has been computerized. The computer program is capable of analyzing reinforced concrete structures of the type described previously in this report and it provides the probability of survival for shelter occupants under various conditions.

The report contains a description of the computer program and a summary of the output data in tabular form. The output data are presented in a form which is suitable for use in the design and construction of shelters.

Appendix I. Summary of the output data in tabular form.

DAMAGE FUNCTIONS FOR UPGRADED SHELTERS

FEMA Contract EMM-C-0374

FINAL REPORT

By

**A. Longinow
M-Y. Wu
J. Mohammadi**

for

**Federal Emergency Management Agency
Washington, D.C. 20472**

August 1982

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20. ABSTRACT

→ The probability of survival is predicted of people located in conventional, expediently upgraded basements when subjected to the blast effects produced by the detonation of a 1-MT weapon near the ground surface. Two categories of potential shelters are considered here, i.e., engineered buildings and single-family residences.

The first category included 12 basements designed for live loads in the range from 50 psf to 250 psf and slab spans from 12 ft to 20 ft. Each of these was analyzed as expediently upgraded using four different upgrading schemes. An expedient upgrading scheme involves strengthening the slab over the basement by providing intermediate supports and blocking off all openings into the basement. This resulted in 60 shelters of different strengths which include the conventional, unupgraded slabs as base cases.

The second category included four conventional single-family dwellings with full basements. Each was evaluated when upgraded using a "studwall" upgrading concept. Two of the basements were reevaluated using the "post and beam" upgrading concept. These upgrading concepts are essentially similar and were used as intermediate supports for strengthening the joist floor systems.

A probability of survival function was developed for each shelter and each upgrading scheme. The procedure used to accomplish this consists of two parts. The first is a probabilistic structural analysis which determines the probability of failure to the shelter envelope. The second is a probabilistic people survival analysis which considers two casualty producing mechanisms, i.e., debris effects from the collapse of the overhead slab and primary blast. The probability of structural failure is made use of in computing the probability of survival against debris effects.

→ The report includes a description of the shelters analyzed, a description of the method used in performing the analysis, detailed results, conclusions and recommendations.

FOREWORD

This is the final report on IIT Research Institute (IITRI) project No. J06528 entitled, "Damage Functions for Upgraded Shelters." It was performed for the Federal Emergency Management Agency (FEMA) under Contract EMW-C-0374. The study was initiated September 15, 1980 and completed on August 20, 1981. The work was performed by Dr. A. Longinow (project engineer and principal investigator) and Mr. Ming-Yeh Wu of the Engineering Mechanics Section, Division M, IITRI, and by Dr. J. Mohammadi of the Department of Civil Engineering, Illinois Institute of Technology, Chicago, Illinois. The study was monitored by Mr. D. A. Bettge of FEMA.

Respectfully submitted,
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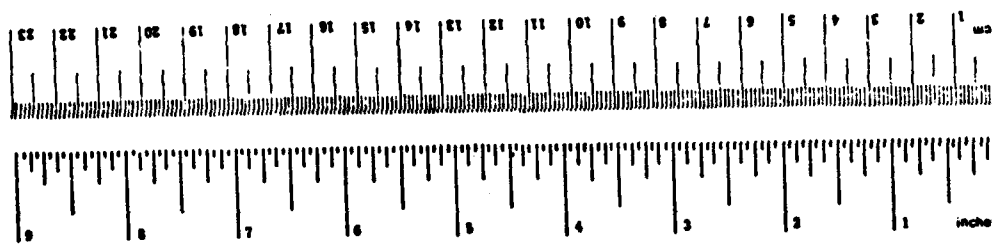
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METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures			
Symbol	When You Know	Multiply by	To Find
LENGTH			
in	inches	2.5	centimeters
ft	feet	30	centimeters
y	yards	0.9	meters
m	miles	1.6	kilometers
AREA			
sq in	square inches	6.5	square centimeters
sq ft	square feet	9.3	square meters
sq yd	square yards	10.8	square meters
sq mi	square miles	2.6	square kilometers
ac	acres	0.4	hectares
MASS (weight)			
oz	ounces	28	grams
lb	pounds	4.5	kilograms
sh	short tons (2000 lb)	0.9	metric tons
VOLUME			
cup	cup	0.24	liters
tblsp	tablespoon	0.015	liters
fl oz	fluid ounce	0.03	liters
qt	quart	0.95	liters
pt	pint	0.47	liters
gal	gallon	3.8	liters
cu ft	cubic foot	0.03	cubic meters
cu yd	cubic yard	0.76	cubic meters
TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature



* 1 in = 2.54 cm exactly. For other exact conversions, see metric tables, Pub. 256, Units of Length and Measure, Pub. 257, 50 Cashing, or C.I.T. 206.

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1. INTRODUCTION

Eventually, existing buildings may need to be surveyed and designated for upgrading to provide fallout and blast protection for evacuees and workers of critical industries in the event of a nuclear weapon attack.

This study was concerned with predicting the probability of survival of people located in expediently upgraded conventional basements when subjected to the blast effects of a 1-MT weapon detonated near the ground surface. Two categories of basements are considered, i.e., basements of engineered buildings and basements of single family residences.

The first category refers to low-rise engineered buildings with basements. The first floor is at grade and, therefore, the slab over the basement is directly exposed to the blast load. The basement walls, by virtue of their location, are not directly exposed to the blast. In the analysis performed, the first floor slab is treated as the primary structural component. Its collapse will result in casualties due to debris impact and due to blast loads entering shelter areas when the shelter envelope is breached. Interior and exterior basement walls are assumed to be stronger than the overhead slab and are not explicitly considered in the analysis.

The first floor slab can be most expediently strengthened by reducing its effective span. This can be done by introducing intermediate supports. In this study such supports are applied to the slab but not to the walls. Such supports are referred to as "expedient upgrading" and may consist of timber, steel, and masonry.

The objective here is not to evaluate the particular supports used, but rather to determine the reliability of the shelter when an intermediate support is provided. Other studies have been devoted to the design and experimental evaluation of expedient upgrading schemes (Ref 1,2).

The first floor slab was designed (Ref 3) as a one-way system for live loads in the range from 50 psf to 250 psf. This represents a fairly wide range of use classes. The lower bound applies to classrooms and public rooms,

while the upper bound applies to industrial buildings, e.g., light manufacturing and some small warehouses. A total of twelve separate cases representing three different span lengths (12, 16, and 20 ft) and four different design live loads (50, 80, 125, and 250 psf) were considered. Each of the twelve basements was analyzed as expediently upgraded using four different upgrading schemes. This resulted in sixty shelters which include the conventional (un-upgraded) design as the base case. As used in this study, an expedient upgrading scheme involves supporting the first floor slab and blocking off all openings into the basement.

The second category of shelter considered includes four conventional wood frame residences with basements. These are real buildings whose plans were obtained from local engineer/architect offices. Expedient upgrading schemes considered in this portion of the study include the "studwall" and "post and beam." The objective is to reduce the effective span length of the joist floor over the basement. Six shelters were evaluated. First, each basement was evaluated as upgraded using the studwall scheme. Second, two of the basements were reevaluated using the post and beam concept. The process was repeated by assuming that 1 ft of soil would be placed over the first floor for radiation protection. Placing 2 ft of soil would significantly affect the strength of the floor system. The case involving 2 ft of soil was, therefore, not considered.

A probability of survival function was developed for each shelter and each particular upgrading scheme. The method used in determining the probability of people survival is described.

The analysis procedure formulated and used in this study consists of two parts. The first part is a probabilistic structural analysis which determines the probability of shelter failure (collapse). This analysis is capable of considering all structural components and the respective failure modes of each component. For example, in the case of the reinforced concrete slab, both flexure and shear are considered as contributing to collapse. The probability of failure for each mode acting independent of the others is determined first. Correlation between them is not evaluated. The results are then used to determine the upper and lower bounds on the probability of failure for each component and then for the structure as a whole. As an example, see the analysis presented in Appendix A.

The second part is a probabilistic people survival analysis which makes use of the probability of structural failure results. Casualty mechanisms considered include debris from the collapse of the shelter and primary blast. Probability of survival against primary blast is determined on the basis of available casualty data (Ref 4). This report is arranged as follows.

Chapter 2 includes a detailed description of the structural analysis used in predicting the response of reinforced concrete slabs when subjected to blast loading. The corresponding probabilistic analysis is presented in Chapter 3. These two chapters form the basis of a computer program which was started in the previous effort (Ref 5) and then modified and extended in the course of the study reported. This computer program is capable of computing the probability of survival of people located in basement shelters when subjected to blast produced by the detonation of a nuclear weapon. The procedure used is as described in the previous paragraphs of this chapter.

In its present form this computer program can analyze basement shelters in which the roof slab is the primary structural component and the walls are not considered in the analysis. It, therefore, applies to cases in which the walls are not exposed to the blast, or by virtue of their design and location are substantially stronger than the slab. The program consists of two separate parts, which treat the following problems:

- (1) Basements with two-way roof slabs and with membrane resistance along two or four opposite edges. In addition to membrane resistance, four support conditions may be considered.
 - (a) All edges simply supported
 - (b) All edges fixed (clamped)
 - (c) Long edges simply supported, short edges fixed
 - (d) Short edges simply supported, long edges fixed.
- (2) Basements with two-way roof slabs, without membrane resistance. These four support conditions may be considered.
 - (a) All edges simply supported
 - (b) All edges fixed
 - (c) Long edges simply supported, short edges fixed
 - (d) Short edges simply supported, long edges fixed.

The computer program computes the probability of survival for people located in basements when subjected to blast effects produced by the detonation of a nuclear weapon in its Mach region. Megaton or kiloton weapons may be specified. The computer program has some of the following features.

- (1) It predicts the upper and lower bounds on the probability of component collapse. In doing this both the flexural and shear modes of failure are considered.
- (2) Predicts the probability of people survival based on:
 - (a) Debris effects from the collapse of the overhead slab
 - (b) Blast pressures due to primary blast.Slab collapse modes on which the debris effects are based were estimated based on review of experimental data.
- (3) Considers statistical variability in the following parameters:
Blast load parameters - F_1 , t_d
Structure parameters - A_s , A'_s , f'_c , f_y , d , d' , ϕ

where F_1 = peak overpressure

t_d = positive phase duration

A_s = tension steel

A'_s = compression steel

f'_c = compressive concrete strength

f_y = reinforcement yield strength

ϕ = undercapacity factor in bending.

It is our considered opinion that this computer program is superior to any that exist in related areas. The reasons are:

- (1) The program analyzes actual structures and makes predictions on the basis of analytic results. No scaling is involved.
- (2) Parameter variability is considered in more detail and on a larger scale than other methods (such as the FAST code, Reference 19, for example). The results, therefore, are more reliable.
- (3) The program is capable of evaluating the effectiveness of different expedient upgrading schemes on people survivability.

- (4) The program does not use a simulation approach, such as Monte Carlo, for example, and is, therefore, quick and economical in computer usage.
- (5) The program is oriented specifically to the civil defense (national security) problem.

This computer program can and should be extended to include a more complete set of structural components, i.e., walls, columns, and girders. This would extend its applicability to a wider class of structures and would thus increase its utility.

For the sake of clarity and generality in presentation, the structural analysis procedure given in Chapter 2 and the probabilistic analysis procedure given in Chapter 3 are explained with reference to a square, two-way slab fixed along the edges.

Reinforced concrete shelters considered here are described in Chapter 4 which also includes a description of the expedient upgrading options considered. People survivability results are summarized in Chapter 5. Conclusions and recommendations are presented in Chapter 5 together with a short summary of this study.

Analysis of residential basements is presented in Appendices A and B. Appendix C contains detailed probability of failure and probability of survival results for the reinforced concrete basement shelters.

2. STRUCTURAL ANALYSIS

2.1 STRUCTURE IDEALIZATION AND LOADING

The general form of the structural analysis performed in this study is described in Reference 6 and discussed in relation to the structure shown in Figure 1. This is a portion of a conventional basement which serves as a personnel shelter against the effects of blast produced by a nuclear weapon detonated near the ground surface. With the entranceways and other openings into the basement blocked off, the structural component of primary interest is the first floor slab. Its collapse will result in casualties due to slab debris impact and due to blast pressures and velocities penetrating basement areas where people would be located. Basement walls, both interior and exterior (peripheral), are not expected to fail prior to the failure of the first floor slab and are, therefore, not considered in the analysis.

The roof (first floor) slab is modeled as a single degree of freedom system whose resistance is a piecewise linear function. The point at which response is sought is at the center of the slab. We are interested in its peak deflection when the slab is subjected to a time dependent load over its surface. We are also interested in the peak dynamic reactions distributed along the edges of the slab. The blast load is approximated using the following expression (Ref 7):

$$F(t) = F_1 \left(1 - \frac{t}{t_d}\right) e^{-t/t_d} \quad (1)$$

where F_1 = peak load magnitude

t_d = positive phase duration of the blast load.

The spatial distribution of the blast load is assumed to be uniform over the surface of the slab. The interaction of the blast wave with the building

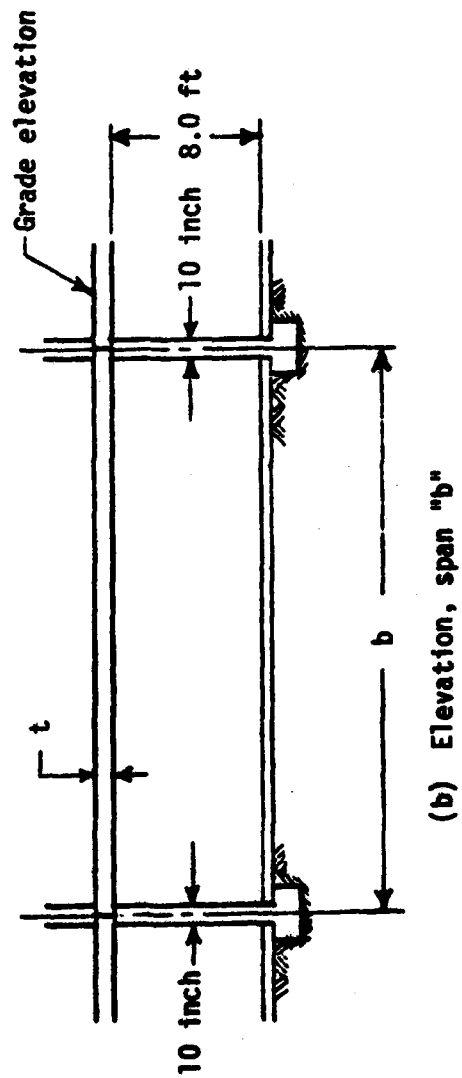
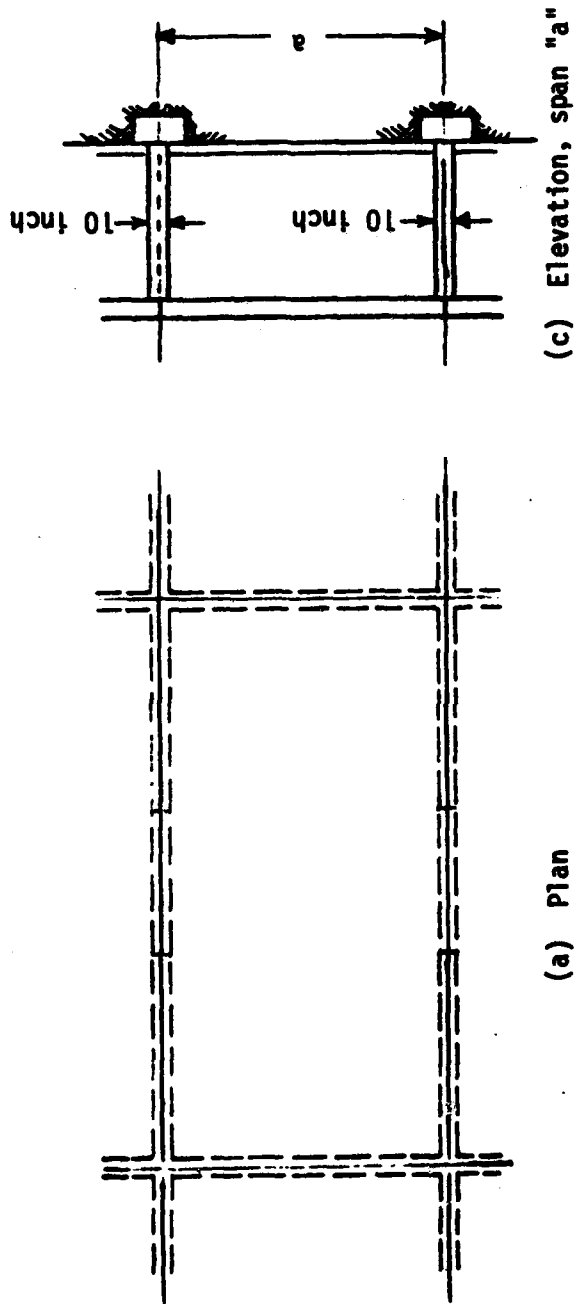


Figure 1. Basement shelter, plan and elevation views.

above the basement is assumed not to modify the free-field character of the blast wave to any significant extent. Therefore, equation (1) need not be modified for this effect.

2.2 RESISTANCE FUNCTION

A resistance function for uniformly loaded, two-way reinforced concrete slabs fixed along the edges is shown in Figure 2. The maximum resistance in the elastic range, R_1 is assumed to be developed when the most highly stressed section reaches its plastic resistance. For slabs fixed along the edges this section is along the long edges. For a square, clamped slab for example, R_1 is (Ref 6)

$$R_1 = 29.2 M_{ub}^o \quad (2)$$

where M_{ub}^o = negative ultimate moment capacity per unit width at the center of the long edge.

The maximum resistance in the elasto-plastic range, R_2 , is determined on the assumption that the ultimate bending moment is developed along all yield lines representing a minimum load yield pattern. Thus for a square clamped slab (Ref 6)

$$R_2 = \frac{12}{a} (M_{u1} + M_{u2} + M_{u3} + M_{u4}) \quad (3)$$

where M_{u1} = total ultimate moment capacity along midspan section parallel to edge "a"

M_{u2} = total negative ultimate moment capacity along edge "a"

M_{u3} = total ultimate moment capacity along midspan section parallel to edge "b"

M_{u4} = total negative ultimate moment capacity along edge "b" (see Figure 3 for distribution of ultimate moments)

a = span length in the short direction.

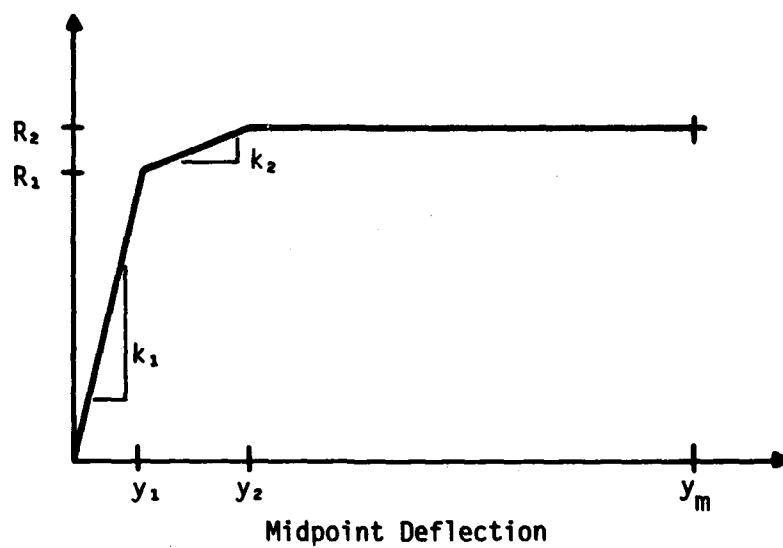


Figure 2. Resistance function for a two-way slab.

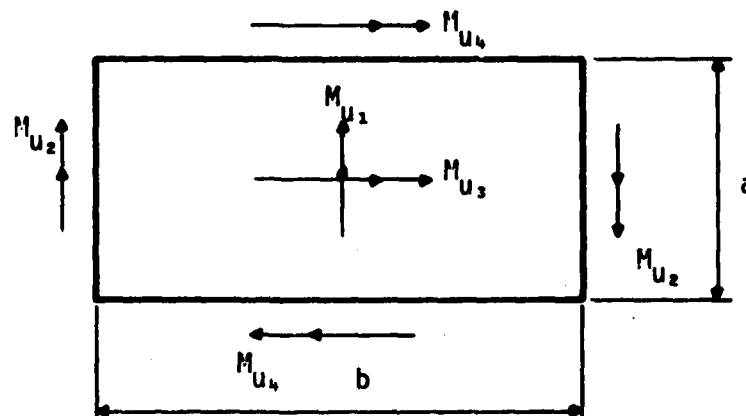


Figure 3. Distribution of ultimate moments.

The midpoint deflection y_1 is

$$y_1 = \frac{R_1}{k_1} \quad (4)$$

where k_1 , the slab stiffness in the elastic range for a square, clamped slab for example, is (Ref 6)

$$k_1 = \frac{810 E I_a}{a^2} \quad (5)$$

where E = modulus of elasticity of concrete

I_a = moment of inertia of a unit width of slab.

$$E = 33w^{3/2} \sqrt{f'_c} \quad (6)$$

where w = unit weight of concrete, lb/cu ft

f'_c = ultimate compressive strength of concrete, psi.

$$I_a = \frac{bd^3}{2} (5.5 \rho_a + 0.083) \quad (7)$$

where ρ_a = average reinforcement ratio, for a slab with uniform reinforcement, $\rho_a = A_s/bd$, where A_s is the steel area per unit width, b , and d is the effective depth of the section.

$$y_2 = y_1 + \frac{R_2 - R_1}{k_2} \quad (8)$$

where k_2 , the slab stiffness in the elasto-plastic range also for a square, clamped slab, is (Ref 6)

$$k_2 = \frac{252 E I_a}{a^2} \quad (9)$$

Based on an examination of experimental results, Reference 8 recommends that the failure (incipient collapse) deflection, y_m , be computed as

$$y_m = 0.15a \quad (10)$$

2.3 DYNAMIC REACTIONS

The dynamic reactions along the edges of two-way slabs are determined on the basis of the assumption that the distribution of the inertial forces is the same as the assumed deflected shape of the slab and the resistance is uniformly distributed.

For a fixed, uniformly loaded two-way slab with $a/b = 0.5$ for example, the total dynamic reaction, V_a , at edge "a" in the elastic range is (Ref 6)

$$V_a = 0.05P + 0.08R \quad (11)$$

The corresponding dynamic reaction at edge "b" is

$$V_b = 0.12P + 0.25R \quad (12)$$

In the elasto-plastic range

$$V_a = 0.04P + 0.09R \quad (13)$$

$$V_b = 0.09P + 0.28R \quad (14)$$

In the plastic range

$$V_a = 0.04P + 0.08R_2 \quad (15)$$

$$V_b = 0.11P + 0.27R_2 \quad (16)$$

where

$$P = abF_1(t) \quad (17)$$

$$R = R(y) \quad (18)$$

It was assumed in this study that the dynamic reactions are uniformly distributed along the respective edges (Figure 4). The critical shear stress, \bar{v}_u , was computed at a section, $d/2$, from the face of support using the approximation shown in Figure 5. Thus, the critical shear stress along edge "b" is

$$\bar{v}_{ub} = \frac{v_{ub}}{a} (a - t_w - d) \quad (19)$$

$$\bar{v}_{ua} = \frac{v_{ua}}{b} (b - t_w - d) \quad (20)$$

where $v_{ub} = \frac{V_b}{bd}$ (21)

$$v_{ua} = \frac{V_a}{ad} \quad (22)$$

t_w = support (wall) thickness

d = effective depth of slab.

2.4 SHEAR RESISTANCE

The shear resistance provided by the concrete can be computed using the following expression (Refs 9,10)

$$V_c = \left(1.9 \sqrt{f'_c} + 2500 \rho_w \frac{V_{ud}}{M_u} \right) bd \quad (23)$$

but not greater than $3.5 bd \sqrt{f'_c}$.

where $\rho_w = A_s/bd$

b = width of section

V_u = the shear at the section

M_u = the bending moment at the section occurring simultaneously with V_u .

The quantity V_{ud}/M_u is not to be taken greater than 1.0 in computing V_c .

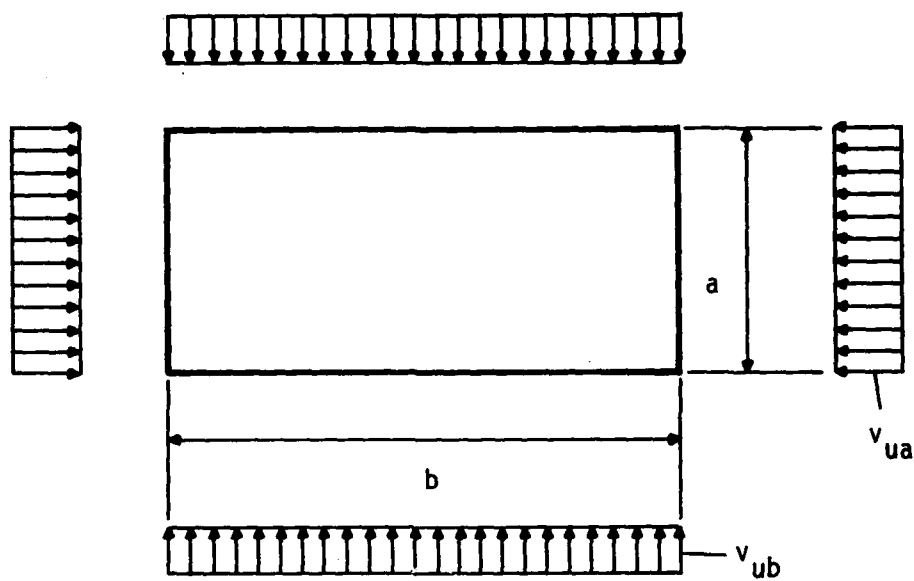


Figure 4. Assumed distribution of dynamic reactions along edges "a" and "b".

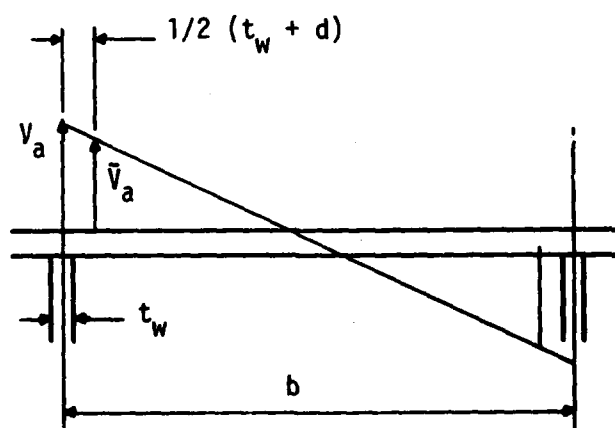


Figure 5. Critical shear stress, \bar{v}_a .

The shear resistance can also be approximated (Ref 10) by

$$V_c = 2 \sqrt{f'_c} bd \quad (24)$$

According to Reference 9, equation (23) is too conservative for predicting structural failure due to shear and recommends that values obtained from equation (23) be increased by 50% when used for this purpose. Consequently, the following expression was used in investigating shear failure of slabs:

$$v_m = 1.5 \frac{V_c}{bd} \quad (25)$$

where v_m = the unit capacity of the slab.

In computing v_m , the ultimate dynamic strength of concrete, f'_{dc} , was used.

$$f'_{dc} = 1.25 f'_c \quad (26)$$

2.5 DYNAMIC RESPONSE ANALYSIS

Since both the loading and resistance are complex functions, it was necessary to use a numerical procedure to obtain the peak midpoint deflection of the slab and the peak dynamic reactions. The equation solved is

$$K_{LM} M_t \ddot{y} + R(y) = F(t) \quad (27)$$

where K_{LM} = the load-mass factor which is used to transform the real system to an equivalent single degree of freedom system. For a square, clamped reinforced concrete slab, K_{LM} has the following values (Ref 6):

Elastic range, $K_{LM} = 0.63$

Elasto-plastic range, $K_{LM} = 0.67$

Plastic range, $K_{LM} = 0.51$

In the membrane range, K_{LM} was taken as 1.0

M_t = the total mass of the slab

$R(y)$ = resistance

$F(t)$ = load-time history, see equation (1).

3. PROBABILISTIC ANALYSIS

The survivability or vulnerability of a structure to a given load is a matter of available resistance relative to the imposed load. If the load and resistance could be specified exactly, there would be no question about predicting survivability. However, due to uncertainties, neither the load nor the resistance can be specified precisely and for this reason survivability needs to be expressed in terms of a probability.

For a structure with resistance R and load S , where R and S are random variables, survival is the event $R > S$ and conversely, failure is the event $R < S$. If f_S and f_R are respectively the probability density functions of applied load and resistance, then the probability of failure, $P(F)$, may be related to the overlapping region between f_S and f_R (see Figure 6). Accordingly, the probability of failure is a function of the relative position between μ_S and μ_R (see Figure 6) where μ_S and μ_R are the expected values of S and R , respectively. The probability of failure also depends on the degree of uncertainty (dispersion) in R and S as shown in Figure 7.

Uncertainties arise due to variability in each of the load and resistance parameters, and due to imperfections in the analytic models used in calculating load and resistance.

3.1 PROBABILISTIC ANALYSIS APPROACH

The previous discussion points out the importance of treating the problem of survivability evaluation in probabilistic terms. The corresponding general framework for doing this is described.

For a given structure, its performance function Z can be defined as

$$Z = R - S \quad (28)$$

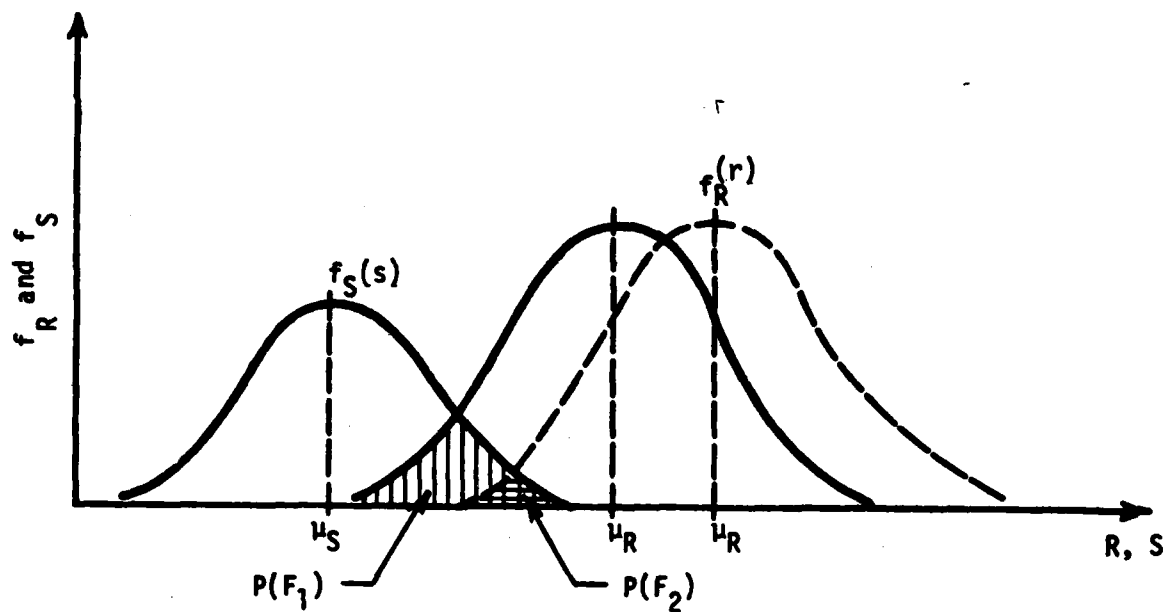


Figure 6. Effect of relative positions of $f_S^{(s)}$ and $f_R^{(r)}$ on $P(F)$.

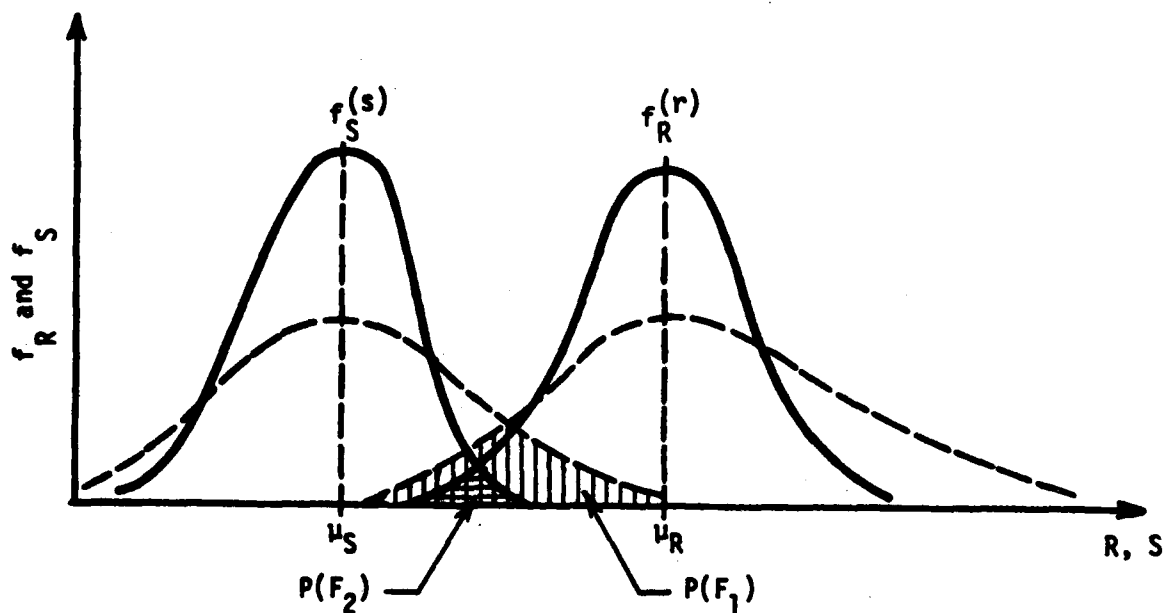


Figure 7. Effect of dispersion of $f_S^{(s)}$ and $f_R^{(r)}$ on $P(F)$.

R and S are respectively functions of several variables and, therefore, also Z, thus

$$Z = g(X_1, X_2, \dots, X_N) \quad (29)$$

The performance limit, i.e., the minimum level of performance that is required for survival, can be set at $Z = z_0$. If $z_0 = 0$, then $Z \leq z_0$ defines a failure state and $Z > z_0$ defines a survival state. Accordingly, the probability of survival, $P(\bar{F})$ is

$$P(\bar{F}) = P(Z > z_0) \quad (30)$$

and the probability of failure, $P(F)$, is

$$P(F) = 1 - P(\bar{F}) = P(Z \leq z_0) \quad (31)$$

If the probability distribution (probability density function) of Z is known, then the probability of failure would be

$$P(F) = \int_{-\infty}^{z_0} f_Z(z) dz = F_Z(z_0) \quad (32)$$

Consider the case in which R and S are known to be lognormal with mean (expected) values μ_R and μ_S and coefficients of variation Ω_R and Ω_S . Assume also that

- (1) R and S are statistically independent
- (2) The probability distributions on R and S are known
- (3) The uncertainty is due only to randomness as described in the distribution $f_Z(z)$
- (4) The performance limit $z_0 = 0$.

For this case, the performance function is

$$Z = \ln(R/S) \quad (33)$$

Since R and S are lognormal variates, their logarithms are normal variates and thus $Z = \ln(R/S) = \ln R - \ln S$ is also a normal variate. Therefore, the probability of failure is

$$P(F) = F_Z(z_0) = \Phi\left(\frac{z_0 - \mu_Z}{\sigma_Z}\right) \quad (34)$$

where $\Phi(\)$ = cumulative density function of the normal distribution

μ_Z = mean (expected) value of Z

σ_Z = standard deviation of Z.

The expected value and standard deviation of Z are obtained as (Ref 11)

$$\mu_Z = \left[\ln \mu_R - \frac{1}{2} \ln(1 + \Omega_R^2) \right] - \left[\ln \mu_S - \frac{1}{2} \ln(1 + \Omega_S^2) \right] \quad (35)$$

where μ_R, μ_S = mean values of R and S, respectively

Ω_R, Ω_S = coefficients of variation of R and S, respectively.

Rearranging R and S, μ_Z becomes

$$\mu_Z = \ln \left[\frac{\mu_R \sqrt{1 + \Omega_S^2}}{\mu_S \sqrt{1 + \Omega_R^2}} \right] \quad (36)$$

$$\sigma_Z^2 = V(Z) = V(\ln R) + V(\ln S) = \ln(1 + \Omega_R^2) + \ln(1 + \Omega_S^2) \quad (37)$$

where $V(\)$ = variance of respective parameter

$$\sigma_Z = \sqrt{\ln \left[(1 + \Omega_R^2)(1 + \Omega_S^2) \right]} \quad (38)$$

Substituting μ_Z and σ_Z into equation (34), $P(F)$ becomes

$$P(F) = \Phi \left[\frac{-\ln \left[\frac{\mu_R \sqrt{1 + \Omega_S^2}}{\mu_S \sqrt{1 + \Omega_R^2}} \right]}{\sqrt{\ln \left[(1 + \Omega_R^2)(1 + \Omega_S^2) \right]}} \right] \quad (39)$$

If Ω_R and Ω_S are not too large, less than about 0.30, then

$$P(F) \approx \Phi \left[\frac{-\ln (\mu_R / \mu_S)}{\sqrt{\Omega_R^2 + \Omega_S^2}} \right] \quad (40)$$

The task of determining $f_Z(z)$ and thus $F_Z(z)$ is generally very involved. The required distribution, $f_Z(z)$, including its parameters, needs to be derived from X_1, X_2, \dots, X_n consistent with equation (29). Given the density functions $f_{X_1}(x_1), f_{X_2}(x_2), \dots, f_{X_n}(x_n)$, the cumulative density function (CDF) of Z for independent X_1, X_2, \dots, X_n would be -

$$F_Z(z) = \int \int \dots \int f_{X_1} f_{X_2} \dots f_{X_n} dx_1 dx_2 \dots dx_n \quad (41)$$

$$\{g(x_1, x_2, \dots, x_n \leq z)\}$$

from which the probability of failure would be obtained.

The effort to derive the exact distribution of Z through equation (41) is clearly laborious and in most cases impractical, because the density functions $f_{X_1}, f_{X_2}, \dots, f_{X_n}$ are usually not known. The information on these variables is generally limited to the mean values and coefficients of variation. However, the necessary type of distribution for $F_Z(z)$ may be prescribed, taking into account relevant physical factors that could contribute to the distribution form with consideration for mathematical tractability.

With regard to the required distribution for Z, we observe the following (Ref 12):

- (1) When $P(F) > 10^{-3}$, the calculated $P(F)$ is approximately the same irrespective of the assumed distribution for Z.
- (2) When $P(F) < 10^{-5}$, the calculated $P(F)$ could be very sensitive to the assumed distribution form.

In the light of these observations, the correct probability of failure may be estimated using any reasonable distribution for Z when $P(F) > 10^{-3}$; whereas for cases where $P(F) < 10^{-5}$, a correct distribution for Z would be necessary to estimate the true risk.

In the case where R and S are not independent, equation (30) would involve the covariance between R and S and the method given in Reference 13 would need to be used.

3.2 ESTIMATING MEANS, VARIANCES, AND COVARIANCES

For the general function of a random variable x, i.e., $y = f(x)$, the mean and variance are

$$E[f(x)] = \int_{-\infty}^{\infty} f(x)g(x)dx \quad (42)$$

$$V[f(x)] = \int_{-\infty}^{\infty} [f(x) - \mu]^2 g(x)dx \quad (43)$$

where $\mu = E[f(x)]$

$g(x)$ = the probability density function.

As discussed earlier in this narrative, in many practical applications $g(x)$ may not be known and information may be limited to the mean and variance of the original variate x. Even if $g(x)$ is known, the integrations indicated by equations (42) and (43) may be difficult to perform. For these reasons, approximate expressions for the mean, variance, and covariance may be obtained

by expanding known functions in Taylor series and neglecting all terms except the linear terms. If R is a function of n variables, i.e.,

$$R = f(x_1, x_2, \dots, x_n) \quad (44)$$

then it can be shown (Ref 14) that the expected value of R is -

$$E(R) = f(\bar{x}_1, \bar{x}_2, \dots, \bar{x}_n) \quad (45)$$

where \bar{x}_i = the mean value.

Also, the variance of R , $V(R)$ is

$$V(R) = \sum_{i=1}^n \left(\frac{\partial R}{\partial x_i} \right)^2 V(x_i) + 2 \sum_{i=1}^{n-1} \sum_{j=2}^n \frac{\partial R}{\partial x_i} \frac{\partial R}{\partial x_j} C(x_i, x_j) \quad (46)$$

where $C(x_i, x_j)$ = covariance between x_i and x_j :

If two functions, R_1 and R_2 , are functions of the same n variables, i.e.,

$$R_1 = f_1(x_1, x_2, \dots, x_n) \quad (47)$$

$$R_2 = f_2(x_1, x_2, \dots, x_n)$$

then the covariance between R_1 and R_2 has the following form (Ref 14):

$$C(R_1, R_2) = \sum_{i=1}^n \frac{\partial R_1}{\partial x_i} \frac{\partial R_2}{\partial x_i} V(x_i) \quad (48)$$

3.3 MULTIPLE FAILURE MODES

If there is more than one mode of failure for a given structural component, the probability of survival will be a function of the respective failure modes. Denoting by R_i the resistance in mode i and by S_i the corresponding load effect, the survival probability is theoretically (Ref 15)

$$P(\bar{F}) = P(R_1 > S_1 \cap R_2 > S_2 \cap \dots \cap R_n > S_n) \quad (49)$$

Equation (49) applies to cases where the failure modes are independent. When the failure modes are highly correlated, the probability of survival becomes

$$P(\bar{F}) = \min P(R_i > S_i) \quad (50)$$

which means that the survival of the component is determined by the weakest mode.

3.4 ANALYSIS OF SYSTEM SURVIVABILITY

The probability of survival described thus far refers to that of a single structural component. The survivability of a complete structure consisting of combinations of such components will depend on the survivabilities of these components and the manner in which they are arranged and connected. The analysis will need to consider the correlation between the components and the degree of redundancy of the system. A procedure for accomplishing this is described in Reference 16.

3.5 MEAN AND VARIANCE OF THE PEAK MIDPOINT DEFLECTION OF THE SLAB

This section contains expressions of the expected value, \bar{y}_p , and the variance, $V(y_p)$ of the peak midpoint deflection, y_p , in the respective ranges of response, i.e., elastic, elasto-plastic, and plastic.

When the magnitude of the blast load is such that the response of the slab is in the elastic range, then (See Fig. 2)

$$R(y) = yR_1/y_1 \quad (51)$$

in accordance with equations (27) and (45) the expected value of the peak midpoint deflection can be expressed as a function of the following parameters, i.e.,

$$\bar{y}_p = f(\bar{F}_1, \bar{t}_d, \bar{R}_1, \bar{y}_1, \bar{M}_t) \quad (52)$$

As indicated previously, \bar{y}_p was determined numerically. The variance of y_p was also determined numerically using the following expression:

$$\begin{aligned} v(y_p) = & \left(\frac{\partial y_p}{\partial F_1} \right)^2 v(F_1) + \left(\frac{\partial y_p}{\partial t_d} \right)^2 v(t_d) + \left(\frac{\partial y_p}{\partial R_1} \right)^2 v(R_1) + \left(\frac{\partial y_p}{\partial y_1} \right)^2 v(y_1) \\ & + \left(\frac{\partial y_p}{\partial M_t} \right)^2 v(M_t) + 2 \left\{ \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial y_1} C(R_1, y_1) + \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial M_t} \right. \\ & \left. C(R_1, M_t) + \frac{\partial y_p}{\partial y_1} \frac{\partial y_p}{\partial M_t} C(y_1, M_t) \right\} \end{aligned} \quad (53)$$

Similarly, when the response is in the elasto-plastic range,

$$R(y) = R_1 + \frac{R_2 - R_1}{y_2 - y_1} (y - y_1) \quad (54)$$

$$\bar{y}_p = f(\bar{F}_1, \bar{t}_d, \bar{R}_1, \bar{R}_2, \bar{y}_1, \bar{y}_2, \bar{M}_t) \quad (55)$$

$$\begin{aligned}
v(y_p) = & \left(\frac{\partial y_p}{\partial F_1} \right)^2 v(F_1) + \left(\frac{\partial y_p}{\partial t_d} \right)^2 v(t_d) + \left(\frac{\partial y_p}{\partial R_1} \right)^2 v(R_1) + \left(\frac{\partial y_p}{\partial R_2} \right)^2 v(R_2) \\
& + \left(\frac{\partial y_p}{\partial y_1} \right)^2 v(y_1) + \left(\frac{\partial y_p}{\partial y_2} \right)^2 v(y_2) + \left(\frac{\partial y_p}{\partial M_t} \right)^2 v(M_t) \\
& + 2 \left\{ \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial R_2} C(R_1, R_2) + \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial y_1} C(R_1, y_1) + \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial y_2} C(R_1, y_2) \right. \\
& + \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial M_t} C(R_1, M_t) + \frac{\partial y_p}{\partial R_2} \frac{\partial y_p}{\partial y_1} C(R_2, y_1) + \frac{\partial y_p}{\partial R_2} \frac{\partial y_p}{\partial y_2} C(R_2, y_2) \\
& + \frac{\partial y_p}{\partial R_2} \frac{\partial y_p}{\partial M_t} C(R_2, M_t) + \frac{\partial y_p}{\partial y_1} \frac{\partial y_p}{\partial y_2} C(y_1, y_2) + \frac{\partial y_p}{\partial y_1} \frac{\partial y_p}{\partial M_t} C(y_1, M_t) \\
& \left. + \frac{\partial y_p}{\partial y_2} \frac{\partial y_p}{\partial M_t} C(y_2, M_t) \right\}
\end{aligned} \tag{56}$$

When the response is in the plastic range, $R(y) = R_2$ and

$$\bar{y}_p = f(\bar{F}_1, \bar{t}_d, \bar{R}_1, \bar{R}_2, \bar{y}_1, \bar{y}_2, \bar{y}_m, \bar{M}_t) \tag{57}$$

$$\begin{aligned}
v(y_p) = & \left(\frac{\partial y_p}{\partial F_1} \right)^2 v(F_1) + \left(\frac{\partial y_p}{\partial t_d} \right)^2 v(t_d) + \left(\frac{\partial y_p}{\partial R_1} \right)^2 v(R_1) + \left(\frac{\partial y_p}{\partial R_2} \right)^2 v(R_2) \\
& + \left(\frac{\partial y_p}{\partial y_1} \right)^2 v(y_1) + \left(\frac{\partial y_p}{\partial y_2} \right)^2 v(y_2) + \left(\frac{\partial y_p}{\partial y_m} \right)^2 v(y_m) + \left(\frac{\partial y_p}{\partial M_t} \right)^2 v(M_t) \\
& + 2 \left\{ \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial R_2} C(R_1, R_2) + \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial y_1} C(R_1, y_1) + \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial y_2} C(R_1, y_2) \right. \\
& + \frac{\partial y_p}{\partial R_1} \frac{\partial y_p}{\partial M_t} C(R_1, M_t) + \frac{\partial y_p}{\partial R_2} \frac{\partial y_p}{\partial y_1} C(R_2, y_1) + \frac{\partial y_p}{\partial R_2} \frac{\partial y_p}{\partial y_2} C(R_2, y_2) \\
& + \frac{\partial y_p}{\partial R_2} \frac{\partial y_p}{\partial M_t} C(R_2, M_t) + \frac{\partial y_p}{\partial y_1} \frac{\partial y_p}{\partial y_2} C(y_1, y_2) + \frac{\partial y_p}{\partial y_1} \frac{\partial y_p}{\partial M_t} C(y_1, M_t) \\
& \left. + \frac{\partial y_p}{\partial y_2} \frac{\partial y_p}{\partial M_t} C(y_2, M_t) \right\}
\end{aligned} \tag{58}$$

3.6 MEAN AND VARIANCE OF THE MAXIMUM SHEAR STRESS

This section contains expressions of the expected value, \bar{v}_{up} , and the variance, $V(v_{up})$ of the maximum shear stress in the respective ranges of slab response, i.e., elastic, elasto-plastic, and plastic.

As indicated previously (see Section 2.3), the shear stress in the concrete along a given edge of a slab is computed using equations (19) or (20). When the magnitude of the load is such that response is in the elastic range then the peak shear stress can be expressed as a function of the following parameters, i.e.,

$$\bar{v}_{up} = f(\bar{F}_1, \bar{t}_d, \bar{R}_1, \bar{y}_1, \bar{d}) \quad (59)$$

The magnitude of \bar{v}_{up} was determined numerically for each edge and the maximum of the two was used in the analysis. The variance of v_{up} was also determined numerically using the following expression:

$$\begin{aligned} V(v_u) = & \left(\frac{\partial v_u}{\partial F_1} \right)^2 V(F_1) + \left(\frac{\partial v_u}{\partial t_d} \right)^2 V(t_d) + \left(\frac{\partial v_u}{\partial R_1} \right)^2 V(R_1) + \left(\frac{\partial v_u}{\partial y_1} \right)^2 V(y_1) \\ & + \left(\frac{\partial v_u}{\partial d} \right)^2 V(d) + 2 \left\{ \frac{\partial v_u}{\partial R_1} \frac{\partial v_u}{\partial y_1} C(R_1, y_1) + \frac{\partial v_u}{\partial R_1} \frac{\partial v_u}{\partial d} C(R_1, d) \right. \\ & \left. + \frac{\partial v_u}{\partial y_1} \frac{\partial v_u}{\partial d} C(y_1, d) \right\} \end{aligned} \quad (60)$$

Similarly, when the response is in the elasto-plastic range,

$$\bar{v}_{up} = f(\bar{F}_1, \bar{t}_d, \bar{R}_1, \bar{R}_2, \bar{y}_1, \bar{y}_2, \bar{d}) \quad (61)$$

$$\begin{aligned}
V(v_{up}) = & \left(\frac{\partial v_{up}}{\partial F_1} \right)^2 V(F_1) + \left(\frac{\partial v_{up}}{\partial t_d} \right)^2 V(t_d) + \left(\frac{\partial v_{up}}{\partial R_1} \right)^2 V(R_1) + \left(\frac{\partial v_{up}}{\partial R_2} \right)^2 V(R_2) \\
& + \left(\frac{\partial v_{up}}{\partial y_1} \right)^2 V(y_1) + \left(\frac{\partial v_{up}}{\partial R_2} \right)^2 V(R_2) + \left(\frac{\partial v_{up}}{\partial y_2} \right)^2 V(y_2) + \left(\frac{\partial v_{up}}{\partial d} \right)^2 V(d) \\
& + 2 \left\{ \frac{\partial v_{up}}{\partial R_1} \frac{\partial v_{up}}{\partial R_2} C(R_1, R_2) + \frac{\partial v_{up}}{\partial R_1} \frac{\partial v_{up}}{\partial y_1} C(R_1, y_1) + \frac{\partial v_{up}}{\partial R_1} \frac{\partial v_{up}}{\partial y_2} C(R_1, y_2) \right. \\
& + \frac{\partial v_{up}}{\partial R_1} \frac{\partial v_{up}}{\partial d} C(R_1, d) + \frac{\partial v_{up}}{\partial R_2} \frac{\partial v_{up}}{\partial y_1} C(R_2, y_1) + \frac{\partial v_{up}}{\partial R_2} \frac{\partial v_{up}}{\partial y_2} C(R_2, y_2) \\
& + \frac{\partial v_{up}}{\partial R_2} \frac{\partial v_{up}}{\partial d} C(R_2, d) + \frac{\partial v_{up}}{\partial y_1} \frac{\partial v_{up}}{\partial y_2} C(y_1, y_2) + \frac{\partial v_{up}}{\partial y_1} \frac{\partial v_{up}}{\partial d} C(y_1, d) \\
& \left. + \frac{\partial v_{up}}{\partial y_2} \frac{\partial v_{up}}{\partial d} C(y_2, d) \right\} \quad (62)
\end{aligned}$$

When the response is in the plastic range,

$$\bar{v}_{up} = f(\bar{F}_1, \bar{t}_d, \bar{R}_2, \bar{d}) \quad (63)$$

$$\begin{aligned}
V(v_{up}) = & \left(\frac{\partial v_{up}}{\partial F_1} \right)^2 V(F_1) + \left(\frac{\partial v_{up}}{\partial t_d} \right)^2 V(t_d) + \left(\frac{\partial v_{up}}{\partial R_2} \right)^2 V(R_2) \\
& + \left(\frac{\partial v_{up}}{\partial d} \right)^2 V(d) + 2 \left\{ \frac{\partial v_{up}}{\partial R_2} \frac{\partial v_{up}}{\partial d} C(R_2, d) \right\} \quad (64)
\end{aligned}$$

In the above expressions the blast load parameters, F_1 and t_d , are independent of the resistance function parameters R_1 , R_2 , y_1 , y_2 , y_m , and the mass of the slab, M_t . Although t_d is a function of F_1 , the covariance using equation (48) turned out to be zero. The ultimate (failure) deflection, y_m , see equation (10), is independent of the other resistance function parameters because the span length "a" is taken as a constant.

In the above expressions for the variances of y_p and v_{up} , the constituent variances and covariances were determined in closed form. The partial

derivatives were obtained by means of numerical differentiation using

$$\frac{\partial y_p}{\partial x_i} = \frac{(y_p)_{i+1} - (y_p)_{i-1}}{2\Delta h} \quad (65)$$

where $x_i = (F_i, t_d, y_m, M_t, Q_t, Q_{ft}, y_{ft})$

Δh = differentiation increment, Δh was taken as 0.10 of one standard deviation for each of the variables.

3.7 PROBABILITY OF SHELTER FAILURE

For the reinforced concrete basement shelters considered in this study (see Figure 1), the first floor (overhead) slab is considered to be the primary structural component. Its collapse will result in casualties. The probability of failure of the slab was determined on the basis of the theory presented in Sections 3.1 to 3.3. Specific expressions used in the analysis are given.

3.7.1 Probability of Slab Failure

As indicated in Section 3.3, in the case of two failure modes, the probability of slab failure, $P(F)$, is

$$P(F) = 1 - [1 - P(F_b)][1 - P(F_v)] \quad (66)$$

if the modes are independent, and

$$P(F) = \max [P(F_b), P(F_v)] \quad (67)$$

if the modes are highly correlated.

In equations (66) and (67), $P(F_b)$ = probability of failure due to bending (flexure), see Section 3.7.2, and $P(F_v)$ = probability of failure due to shear, see Section 3.7.2.

The real failure probability of the slab is between these two cases, i.e.,

$$\max [P(F_b), P(F_v)] \leq P(F) \leq 1 - [1 - P(F_b)][1 - P(F_v)] \quad (68)$$

In the analysis performed in this study, both bounds were calculated and are included with the results.

3.7.2 Probabilities of Failure Due to Bending and Shear

In accordance with equation (39), probabilities of failure due to flexure and shear were computed using the following expressions:

$$P(F_b) = \Phi \left[\frac{-\ln \left[\frac{y_m}{y_p} \frac{\sqrt{1 + \Omega_{y_p}^2}}{\sqrt{1 + \Omega_{y_m}^2}} \right]}{\ln [(1 + \Omega_{y_p}^2)(1 + \Omega_{y_m}^2)]} \right] \quad (69)$$

where y_m and y_p were defined previously, see Section 3.5

Ω_{y_p} = coefficient of variation of y_p

Ω_{y_m} = coefficient of variation of y_m

$$P(F_v) = \Phi \left[\frac{-\ln \left[\frac{v_m}{v_{up}} \frac{\sqrt{1 + \Omega_{v_{up}}^2}}{\sqrt{1 + \Omega_{v_m}^2}} \right]}{\ln [(1 + \Omega_{v_{up}}^2)(1 + \Omega_{v_m}^2)]} \right] \quad (70)$$

where v_m and v_{up} were defined previously, see equation (25) and Section 3.6

$\Omega_{v_{up}}$ = coefficient of variation of v_{up}

Ω_{v_m} = coefficient of variation of v_m .

3.8 PROBABILITY OF PEOPLE SURVIVAL

Casualty mechanisms considered in this study include primary air blast and debris impact due to the collapse of the first floor slab.

Body damage due to primary air blast results when the blast wave engulfs the body. In such a case, movement of different tissue masses produces shear waves which accelerate different organs and different parts of the same organ to different velocities. This results in strains and frequently in ruptures. Air-filled organs such as the lungs are especially susceptible to this type of damage. Data used to estimate the probability of survival from this effect were obtained from Reference 4 and are reproduced in Table 1.

TABLE 1. PROBABILITY OF SURVIVAL FROM PRIMARY BLAST, $P(S_{pb})$
(30 DAYS AFTER EXPOSURE)

<u>Blast Overpressure</u>	<u>Probability of Survival (%)</u>
40	97.6
50	88.0
60	72.0
70	51.0
80	33.0
100	11.0
120	3.0
150	0.6

Probability of survival against debris due to the collapse of the overhead (first floor) slab is determined using the theorem of total probabilities as

$$P(S_{sc}) = P(S|\bar{F})P(\bar{F}) + P(S|F)P(F) \quad (71)$$

where $P(S_{sc})$ = probability of people survival against structural collapse

$P(S|\bar{F})$ = probability of people survival given that the shelter
does not fail (collapse)

$P(\bar{F})$ = probability of shelter (structure) survival

$P(S|F)$ = probability of people survival given that the shelter fails (collapses)

$P(F)$ = probability of shelter collapse = $1 - P(\bar{F})$.

In equation (71) the probability of structural failure, $P(F)$, is determined as described previously. $P(S|\bar{F})$ and $P(S|F)$ are determined as described in the following paragraphs.

Experimental results (Refs 17,18) indicate that two-way reinforced concrete slabs failing under blast loading do not generally break up into separate pieces. The concrete cracks throughout, but most of the large pieces remain loosely connected to the reinforcing steel. On the other hand, one-way slabs failing under blast loading do not remain connected, but rather break up catastrophically (Ref 2).

At overpressures prior to the total collapse of a slab (one-way or two-way), spallation is expected to occur and shelter occupants will be hit by spalled pieces of concrete. Injuries will, therefore, occur; however, these are not expected to be at the fatality level. It is assumed that prior to slab collapse no fatality level casualties are produced by slab debris and thus $P(S|\bar{F}) = 1.0$.

When people are uniformly distributed in all shelter areas and are lying down at the time of the attack, then $P(S|F)$ can be directly related to the floor area unaffected by collapse. In other words, when an overhead slab collapses people in areas affected by the collapsed debris become fatality level casualties while people in areas unaffected by collapsed debris are survivors and/or injured.

For personnel shelters with two-way overhead slabs, a procedure for computing $P(S|F)$ is given in Reference 5. A procedure for computing $P(S|F)$ for shelters with one-way overhead slabs is described next.

Figure 8 shows an expediently upgraded basement shelter. The particular upgrading consists of a series of 6 in. x 10 in. beams and columns which are used to support the existing slab such that four smaller slabs are produced. For this concept, $P(S|F)$ is computed as follows:

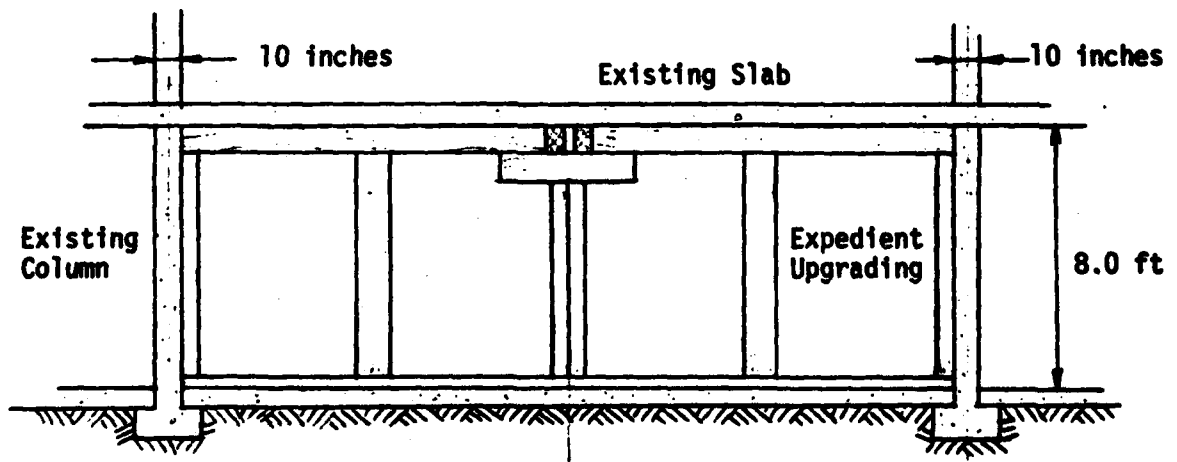
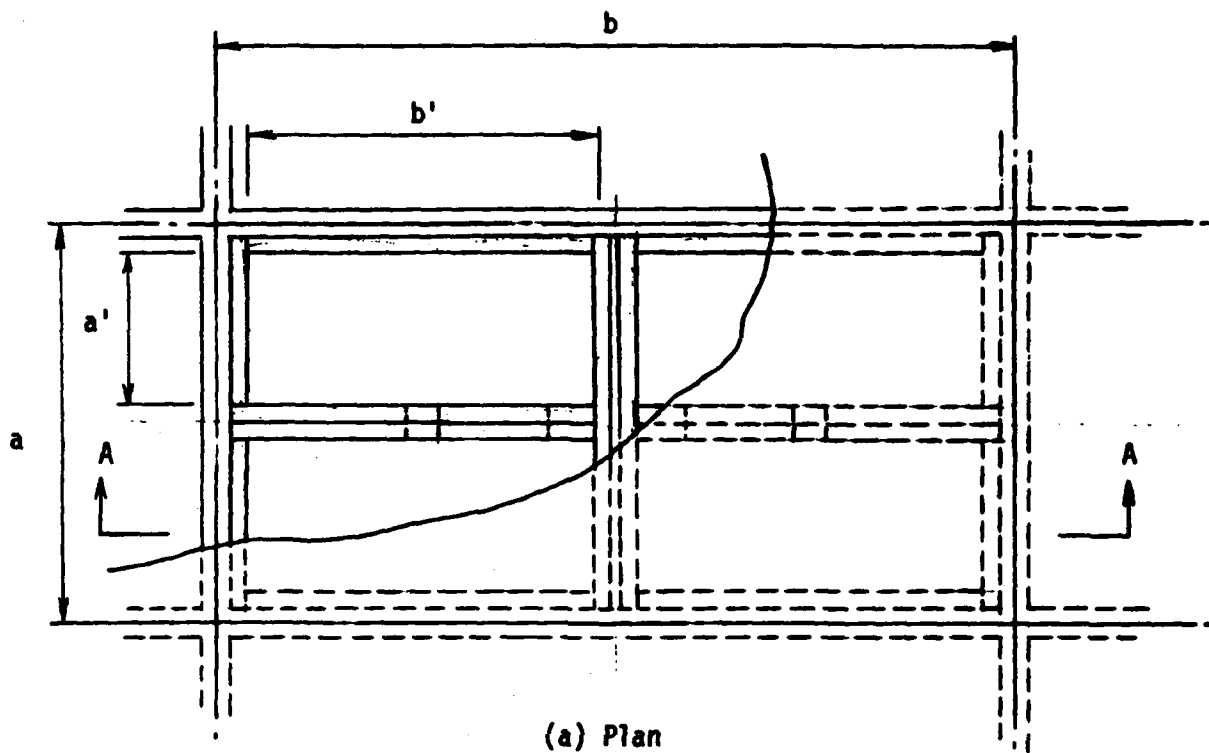


Figure 8. Expedient upgrading, type C.

$$P(S|F) = S_f = \frac{A_f - A_a}{A_f} \quad (72)$$

where A_f = total, clear floor area

A_a = floor area that would be affected by debris from the collapse of the slab.

For the case where $a = 12$ ft and $b = 24$ ft, the spacing of the beams is such that $a' = 4.08$ ft and $b' = 10.08$ ft, then

$$A_f = (12 - 0.83)(24 - 0.83) = 258.81 \text{ sq ft}$$

$$A_a = 4(4.08)(10.08) = 164.51 \text{ sq ft}$$

$$S_f = 0.36.$$

3.9 COMBINED PROBABILITY OF SURVIVAL

It is reasonable to assume that an individual injured from several weapon effects would have a lesser probability of survival than if the injury was due to one of the effects. Information on how the simultaneous action of several effects from a single weapon combine to result in a probability of survival is not known at this time. For this reason, survival probabilities from different effects are combined in this report as independent phenomena. Thus the probability of people survival, $P(S)$, against primary blast and slab collapse is computed as

$$P(S) = P(S_{pb})P(S_{sc}) \quad (73)$$

4. DESCRIPTION OF SHELTERS

4.1 BASIC STRUCTURE

Reinforced concrete shelters considered in this study were illustrated in Figure 1. This is a portion of an engineered basement of a low-rise reinforced concrete building. The first floor slab is a one-way reinforced concrete slab with its top surface at grade. The slab is simply supported along interior reinforced concrete walls. Twelve designs were performed (see Table 2) for several combinations of design live load and span length. The design live load ranges from 50 to 250 psf and the span (short direction) from 12 ft to 20 ft.

4.2 EXPEDIENT UPGRADING SCHEMES

Four types of expedient upgrading schemes were considered with each of the twelve slabs given in Table 2. As illustrated in Figure 9, scheme A is the basic, conventional slab and schemes B through E represent expedient upgrading schemes in the order of increasing strength. Upgrading is accomplished by reducing the basic slab to a series of smaller slabs. This would be done by providing supports along the dash lines shown in Figure 9. Supports that may be used for this purpose were shown in Figure 8.

4.3 ANALYSIS DATA

Table 3 contains structural data used in the analysis of this set of shelters. The various cases and expedient upgrading schemes are identified in Table 2 and Figure 9.

As indicated earlier, the basic slab was designed as a one-way slab. By making use of the temperature reinforcement which is placed orthogonal to the main reinforcement, each slab in each expedient upgrading scheme was analyzed as a two-way slab. In Figure 9, A_{s3} is the main reinforcement and A_{s1} is the temperature reinforcement.

TABLE 2. STRUCTURAL PARAMETERS FOR ONE-WAY SLAB* (REF 3)

Design Parameter	Case											
	1	2	3	4	5	6	7	8	9	10	11	12
Design Live Load, psf	50	50	50	80	80	80	125	125	125	250	250	250
Span, a (ft)	12	16	20	12	16	20	12	16	20	12	16	20
Span, b (ft)	24	32	40	24	32	40	24	32	40	24	32	40
Effective Depth, d (in.)	4.	4.75	7.75	4.50	6.25	7.85	5.22	7.80	10.49	6.74	9.72	12.83
Total Depth, t (in.)	5	5.75	8.75	5.50	7.25	9.0	6.25	9.0	11.50	7.75	10.75	14.0
Main Reinforcement A_{s3} , (in.) ² /ft	0.19	0.23	0.27	0.22	0.26	0.30	0.26	0.35	0.47	0.35	0.47	0.62
Temperature Reinforcement, A_{s1} , (in.) ² /ft	0.11	0.11	0.19	0.11	0.15	0.19	0.13	0.20	0.25	0.16	0.23	0.31

Edge conditions: Simply supported.

Ultimate compressive strength of concrete, $f'_c = 3000$ psi.

Yield strength of reinforcing steel, $f_y = 60,000$ psi.

*See Figure 1 for slab configuration.

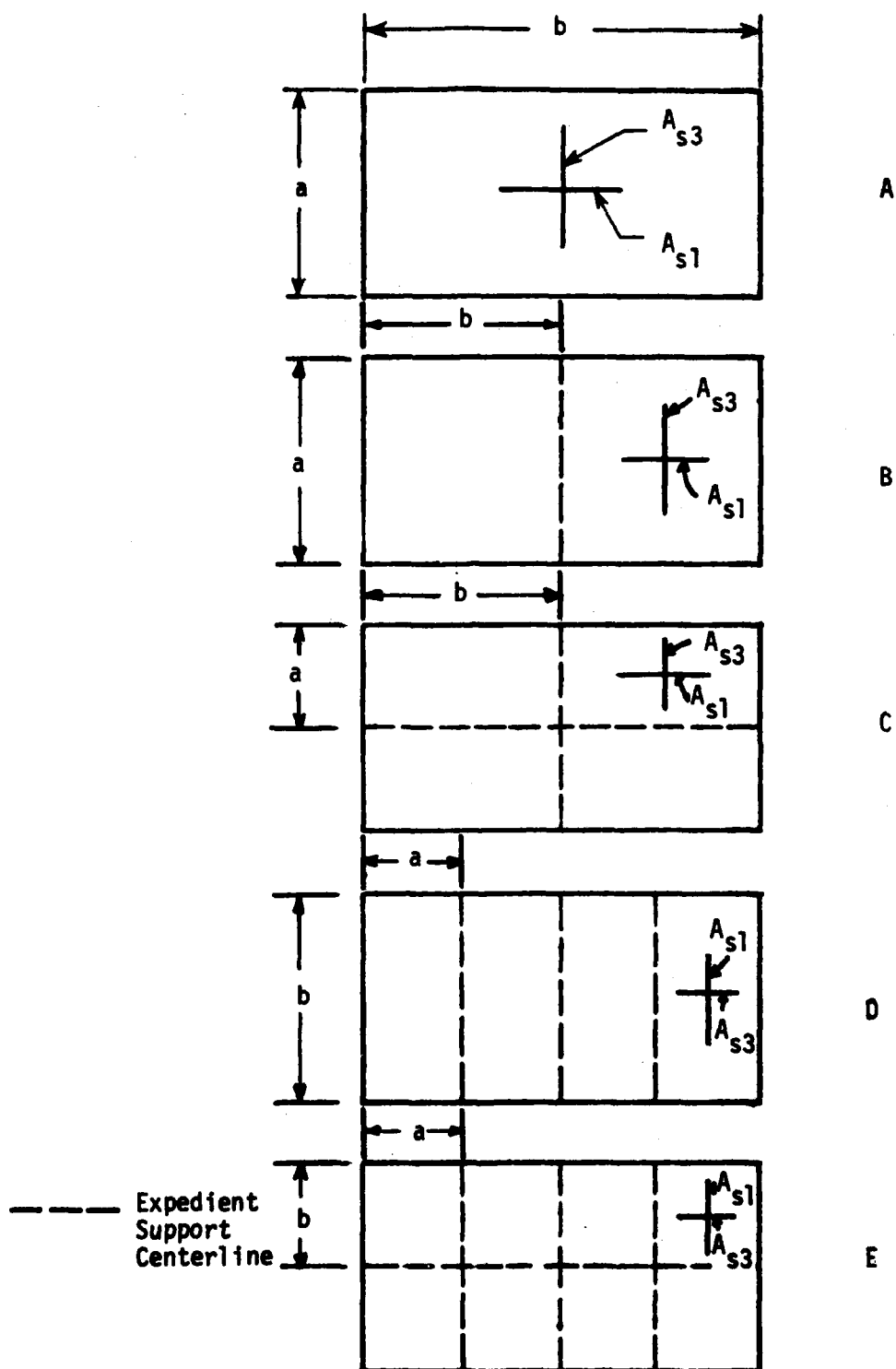


Figure 9. Basic slab and expedient upgrading schemes.

As shown in Table 3, the effective depth of slab, reinforcing steel areas, ultimate strengths of concrete and steel, and the capacity reduction factor are treated as random variables. The other parameters, i.e., span lengths and the slab thickness, are treated as constants. Coefficients of variation were determined based on data given in Chapter 7 of Reference 5. Coefficients of variation of peak overpressure and load duration, see equation (1), were taken as 1.0%.

In Table 3, parameters "a" and "b" are center to center of supports dimensions, while " a_c " and " b_c " are clear span dimensions after the placement of expedient upgrading supports. Values of " a_c " and " b_c " were used in computing S_f by means of equation (72).

In computing S_f for the basic slab (scheme A, Figure 9), A_f and A_a , see equation (72), were assumed not to be the same. In computing A_a , it was assumed that the portion of the floor bounded by the walls and a line 6 in. from the walls would be essentially free from debris effects. Thus for this one case only

$$\left. \begin{aligned} A_f &= (b - 0.83)(a - 0.83) \\ A_a &= (b - 1.83)(a - 1.83) \end{aligned} \right\} \quad (74)$$

TABLE 3. REINFORCED CONCRETE SLAB ANALYSIS DATA

Slab Parameter	Case 1			Case 2			Case 3		
	A	B	E	A	B	E	A	B	E
a (ft)	12.0	12.0	6.0	16.0	16.0	8.0	20.0	20.0	10.0
b (ft)	24.0	12.0	12.0	32.0	16.0	16.0	40.0	20.0	20.0
d (in.)	4.0	4.0	4.0	4.75	4.75	4.75	7.75	7.75	7.75
Ω_d (z)	7.5	7.5	7.5	6.71	6.71	6.71	5.08	5.08	5.08
t (in.)	5.0	5.0	5.0	5.75	5.75	5.75	8.75	8.75	8.75
A_{s1} (in.) ² /ft	0.10	0.10	0.19	0.11	0.11	0.11	0.19	0.19	0.27
A_{s3} (in.) ² /ft	0.19	0.19	0.10	0.23	0.23	0.23	0.27	0.27	0.19
Ω_A (z)	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
ac (ft)	10.17	10.17	4.08	14.17	14.17	6.08	18.17	18.17	8.08
bc (ft)	22.17	10.08	10.17	30.17	14.08	14.17	38.17	18.08	18.17
S_f	0.13	0.21	0.36	0.10	0.16	0.28	0.08	0.12	0.22

Slab Parameter	Case 4			Case 5			Case 6		
	A	B	E	A	B	E	A	B	E
a (ft)	12.0	12.0	6.0	16.0	16.0	8.0	20.0	20.0	10.0
b (ft)	24.0	12.0	12.0	32.0	16.0	16.0	40.0	20.0	20.0
d (in.)	4.50	4.50	4.50	6.25	6.25	6.25	7.85	7.85	7.85
Ω_d (z)	6.94	6.94	6.94	5.7	5.7	5.7	5.05	5.05	5.05
t (in.)	5.5	5.5	5.5	7.25	7.25	7.25	9.0	9.0	9.0
A_{s1} (in.) ² /ft	0.11	0.11	0.22	0.15	0.15	0.15	0.19	0.19	0.30
A_{s3} (in.) ² /ft	0.22	0.22	0.11	0.26	0.26	0.26	0.30	0.30	0.19
Ω_A (z)	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
ac (ft)	10.17	10.17	4.08	14.17	14.17	6.08	18.17	18.17	8.08
bc (ft)	22.17	10.08	10.17	30.17	14.08	14.17	38.17	18.08	18.17
S_f	0.13	0.21	0.36	0.10	0.16	0.28	0.08	0.12	0.22

Ultimate dynamic, compressive strength of concrete, $f'_c = 3.9063$ ksi.Coefficient of variation of f'_c ; $\Omega_c = 17.58\%$.Ultimate dynamic yield strength of steel, $f_{dy} = 84.018$ ksi.Coefficient of variation of f_{dy} ; $\Omega_y = 9.22\%$.Unit weight of concrete, $w = 150$ pcf.Capacity reduction factor, $\phi = 1.0$.Coefficient of variation of ϕ ; $\Omega_\phi = 4.7\%$.

TABLE 3. (Cont.)

Slab Parameter	Case 7					Case 8					Case 9				
	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E
a (ft)	12.0	12.0	6.0	6.0	6.0	16.0	16.0	8.0	8.0	8.0	20.0	20.0	10.0	10.0	10.0
b (ft)	24.0	12.0	12.0	12.0	6.0	32.0	16.0	16.0	16.0	8.0	40.0	20.0	20.0	20.0	10.0
d (in.)	5.22	5.22	5.22	5.22	5.22	7.8	7.8	7.8	7.8	7.8	10.49	10.49	10.49	10.49	10.49
Ω_d (%)	6.33	6.33	6.33	6.33	6.33	5.06	5.06	5.06	5.06	5.06	4.41	4.41	4.41	4.41	4.41
t (in.)	6.25	6.25	6.25	6.25	6.25	9.0	9.0	9.0	9.0	9.0	11.5	11.5	11.5	11.5	11.5
A_{s1} (in.) ² /ft	0.13	0.13	0.13	0.26	0.26	0.20	0.20	0.20	0.35	0.35	0.25	0.25	0.25	0.47	0.47
A_{s3} (in.) ² /ft	0.26	0.26	0.26	0.13	0.13	0.35	0.35	0.35	0.20	0.20	0.47	0.47	0.47	0.25	0.25
ϕ_A (%)	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
a_c (ft)	10.17	10.17	4.08	4.08	4.08	14.17	14.17	6.08	6.08	6.08	18.17	18.17	8.08	8.08	8.08
b_c (ft)	22.17	10.08	10.08	10.17	4.08	30.17	14.08	14.08	14.17	6.08	38.17	18.08	18.08	18.17	8.08
S_f	0.13	0.21	0.36	0.36	0.47	0.10	0.16	0.28	0.28	0.37	0.08	0.12	0.22	0.22	0.29

Slab Parameter	Case 10					Case 11					Case 12				
	A	B	C	D	E	A	B	C	D	E	A	B	C	D	E
a (ft)	12.0	12.0	6.0	6.0	6.0	16.0	16.0	8.0	8.0	8.0	20.0	20.0	10.0	10.0	10.0
b (ft)	24.0	12.0	12.0	12.0	6.0	32.0	16.0	16.0	16.0	8.0	40.0	20.0	20.0	20.0	10.0
d (in.)	6.74	6.74	6.74	6.74	6.74	9.72	9.72	9.72	9.72	9.72	12.83	12.83	12.83	12.83	12.83
Ω_d (%)	5.47	5.47	5.47	5.47	5.47	4.56	4.56	4.56	4.56	4.56	4.06	4.06	4.06	4.06	4.06
t (in.)	7.75	7.75	7.75	7.75	7.75	10.75	10.75	10.75	10.75	10.75	14.0	14.0	14.0	14.0	14.0
A_{s1} (in.) ² /ft	0.16	0.16	0.16	0.35	0.35	0.23	0.23	0.23	0.47	0.47	0.31	0.31	0.31	0.62	0.62
A_{s3} (in.) ² /ft	0.35	0.35	0.35	0.16	0.16	0.47	0.47	0.47	0.23	0.23	0.62	0.62	0.62	0.31	0.61
ϕ_A (%)	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
a_c (ft)	10.17	10.17	4.08	4.08	4.08	14.17	14.17	6.08	6.08	6.08	18.17	18.17	8.08	8.08	8.08
b_c (ft)	22.17	10.08	10.08	10.17	4.08	30.17	14.08	14.08	14.17	6.08	38.17	18.08	18.08	18.17	8.08
S_f	0.13	0.21	0.36	0.36	0.47	0.10	0.16	0.28	0.28	0.37	0.08	0.12	0.22	0.22	0.29

Ultimate dynamic, compressive strength of concrete, $f'_{dc} = 3.9063$ ksi.Ultimate dynamic yield strength of steel, $f_{dy} = 84.018$ ksi.Unit weight of concrete, $w = 150$ pcf.Capacity reduction factor, $\phi = 1.0$.Coefficient of variation of f'_{dc} ; $\Omega_c = 17.58\%$.Coefficient of variation of f_{dy} ; $\Omega_y = 9.22\%$.Coefficient of variation of ϕ ; $\Omega_\phi = 4.7\%$.

5. PEOPLE SURVIVABILITY RESULTS

This chapter contains a summary of results on the probability of survival of people when located in expediently upgraded, reinforced concrete basements and subjected to the blast effects of a 1-MT surface burst. Casualty mechanisms include debris from the collapse of the overhead slab and primary blast. Results were generated using a computer program which was developed during the previous study (Ref 5) and modified and extended in the course of this study. Specific shelters considered are described in Chapter 4.

This chapter explains how the results were produced. Case 1A is used for illustration. The results are summarized in Table 4. A complete set of detailed results is included in Appendix C.

Case 1A (see Table 3) is the basic conventional, one-way slab over the basement designed to resist its own weight plus a live load of 50 psf. Its plan dimensions are 12 ft by 24 ft and the total thickness is 5.0 in. When subjected to a uniformly distributed blast loading over its surface, the possible failure modes are flexure and shear. Probabilities of failure based on flexure and shear when acting independent of each other are shown in Figure 10. Note that shear and flexure are both important in producing failure and, therefore, both need to be considered. However, since we do not know how these two failure modes correlate then the best that can be done is to bound the actual failure probability as discussed in Sections 3.3 and 3.7.1. Thus, using equations (66) and (67), the upper and lower bounds on the failure probability for this slab were computed and are shown in Figure 11. Corresponding bounds on the probability of people survival are shown in Figure 12. They were determined using equation (71).

It is useful to compare the effectiveness of the various expedient upgrading schemes on people survivability. This is done for Case 1 in Table 5 which contains overpressure ranges at the 90% and 50% probability of survival levels.

TABLE 4. SUMMARY OF RESULTS, REINFORCED CONCRETE BASEMENT SHELTERS

CASE	1	2	3	4	5	6	7	8	9	10	11	12	13
	a (ft)	b (ft)	d (in)	t (in)	ea1 sqin/ft	ea3 sqin/ft	Sf	Prob. of Failure 10% 50% 90%	Prob. of People Survival 50% 10%	Upper bound, Lower bound 10% 50%	Upper bound, Lower bound 50% 10%	Upper bound, Lower bound 10% 50%	Upper bound, Lower bound 50% 10%
1A2	12	24	4.00	5.00	0.100	0.190	0.13	0.3475 2.6674	2.4214	3.7174	2.6355 0.3886	3.3020 2.7735	---
1A3	12	24	4.00	5.00	0.100	0.190	0.13	---	---	---	---	---	---
1B2	12	12	4.00	5.00	0.100	0.190	0.21	0.4413 4.2493	3.8618	5.4216	4.1761 0.5502	5.5535 4.5669	---
1B3	12	12	4.00	5.00	0.100	0.190	0.21	---	---	---	---	---	---
1C2	6	12	4.00	5.00	0.100	0.190	0.36	0.6067 12.0363	3.5971	13.5680	11.7252 0.8693	13.0697 12.4713	---
1C3	6	12	4.00	5.00	0.100	0.190	0.36	---	---	---	---	---	---
1D2	6	12	4.00	5.00	0.100	0.190	0.36	0.6017 10.6486	3.5682	12.1021	10.4177 0.8625	11.5696 11.2096	---
1D3	6	12	4.00	5.00	0.100	0.190	0.36	---	---	---	---	---	---
1E2	6	6	4.00	5.00	0.100	0.190	0.47	0.9079 18.7855	5.5580	20.5761	18.2087 1.5778	19.9001 21.1852	---
1E3	6	6	4.00	5.00	0.100	0.190	0.47	---	---	---	---	---	---
2A2	16	32	4.75	5.00	0.110	0.230	0.10	0.3049 2.0553	2.0632	3.1887	2.0329 0.3334	3.9968 2.2291	---
2A3	16	32	4.75	5.00	0.110	0.230	0.10	---	---	---	---	---	---
2B2	16	16	4.75	5.00	0.110	0.230	0.16	0.4633 3.2526	3.1880	4.4858	3.1979 0.5443	4.9125 3.5710	---
2B3	16	16	4.75	5.00	0.110	0.230	0.16	---	---	---	---	---	---
2C2	8	16	4.75	5.00	0.110	0.230	0.28	0.6489 9.4218	3.7067 13.8734	11.0625	9.2065 0.8437	10.5324 9.2541	---
2C3	8	16	4.75	5.00	0.110	0.230	0.28	---	---	---	---	---	---
2D2	8	8	4.75	5.00	0.230	0.110	0.28	0.7045 8.1783	3.8870	9.7293	8.0431 0.9159	9.1341 8.3783	---
2D3	8	8	4.75	5.00	0.230	0.110	0.28	---	---	---	---	---	---
2E2	8	8	4.75	5.00	0.230	0.110	0.37	0.8838 14.5411	4.8094	16.4177	14.1976 1.2909	15.7225 15.3091	---
2E3	8	8	4.75	5.00	0.230	0.110	0.37	---	---	---	---	---	---
3A2	20	40	7.75	8.75	0.190	0.270	0.08	0.4393 2.8711	2.9200	5.1317	2.8425 0.4734	6.3995 3.1131	6.9284
3A3	20	40	7.75	8.75	0.190	0.270	0.08	---	---	---	---	---	---
3B2	20	20	7.75	8.75	0.190	0.270	0.12	0.6410 4.8114	4.6237 12.7456	7.1372	4.7463 0.7134	6.5316 5.0898	---
3B3	20	20	7.75	8.75	0.190	0.270	0.12	---	---	---	---	---	---
3C2	10	20	7.75	8.75	0.190	0.270	0.22	0.7373 13.3486	4.5570 16.1379	16.3247	13.1202 0.8986	15.3703 9.2594	---
3C3	10	20	7.75	8.75	0.190	0.270	0.22	---	---	---	---	---	---
3D2	10	10	7.75	8.75	0.270	0.190	0.22	0.6947 12.4745	3.8862 15.1216	15.3508	12.2459 0.8471	14.3151 10.8465	---
3D3	10	10	7.75	8.75	0.270	0.190	0.22	---	---	---	---	---	---
3E2	10	10	7.75	8.75	0.270	0.190	0.29	0.8379 21.3933	5.0458 30.6936	24.5941	21.0344 1.1051	23.5700 21.2451	---
3E3	10	10	7.75	8.75	0.270	0.190	0.29	---	---	---	---	---	---

Note: Columns 8-13 are overpressures at corresponding probabilities of structural collapse and people survival. Columns 8, 9 and 10 are upper bound values. Columns 11, 12 and 13 are lower bound values. Case numbers (column 1) ending with a "2" refer to the probability of failure of the shelter. Case numbers ending with a "3" refer to the probability of people survival for the same case.

TABLE 4. SUMMARY OF RESULTS, REINFORCED CONCRETE BASEMENT SHELTERS (continued)

CASE	B (ft)	b (ft)	d (in)	λ (in)	λ_{sl} eqin/lb	λ_{s2} eqin/lb	sf	Probability of Failure (Upper bound, Lower bound)			
								10%	50%	90%	90%
402	12	24	4.50	5.50	0.110	0.230	0.13	0.2801	2.2072	4.9037	3.4882
403	12	24	4.50	5.50	0.110	0.220	0.13	3.5323	6.7164	—	6.4350
408	12	12	4.50	5.50	0.110	0.230	0.21	0.6345	3.8430	6.9844	5.5311
403	12	12	4.50	5.50	0.110	0.220	0.21	5.5968	—	—	0.7656
422	6	12	4.50	5.50	0.110	0.230	0.36	0.8744	5.2503	17.4028	15.2275
403	6	12	4.50	5.50	0.110	0.220	0.36	15.5941	—	—	1.2605
402	6	12	4.50	5.50	0.220	0.110	0.36	0.7852	4.7128	15.4102	13.4007
403	6	12	4.50	5.50	0.220	0.110	0.36	13.6080	—	—	1.1300
422	6	6	4.50	5.50	0.220	0.110	0.47	0.9707	5.2650	26.3240	23.5315
403	6	6	4.50	5.50	0.220	0.110	0.47	24.1587	—	—	1.6508
502	16	32	6.25	7.25	0.150	0.260	0.10	0.4108	2.9827	5.8134	3.3283
503	16	32	6.25	7.25	0.150	0.250	0.10	3.3746	8.0187	—	0.4089
508	16	16	6.25	7.25	0.150	0.260	0.16	0.7010	4.6420	7.3624	5.3773
503	16	16	6.25	7.25	0.150	0.250	0.16	5.4844	—	—	0.8061
502	8	16	6.25	7.25	0.150	0.260	0.20	0.8092	4.4417	17.5069	14.8501
503	8	16	6.25	7.25	0.150	0.250	0.20	15.1473	17.8645	—	1.0537
508	8	8	6.25	7.25	0.260	0.150	0.20	0.7401	4.3436	15.9473	13.3003
503	8	8	6.25	7.25	0.260	0.150	0.20	13.6436	16.9206	—	0.9008
522	8	8	6.25	7.25	0.260	0.150	0.37	0.9007	4.9656	26.5405	23.2781
503	8	8	6.25	7.25	0.260	0.150	0.37	23.7692	—	—	1.3234
502	20	40	7.00	9.00	0.190	0.300	0.08	0.4375	3.0003	5.0000	3.1250
503	20	40	7.00	9.00	0.190	0.300	0.08	3.1064	10.7506	—	0.4714
508	20	20	7.00	9.00	0.190	0.300	0.12	0.6082	4.7823	7.7833	5.1520
503	20	20	7.00	9.00	0.190	0.300	0.12	5.2144	12.2256	—	0.7432
522	10	20	7.00	9.00	0.190	0.300	0.22	0.7787	4.3309	17.0085	14.0046
503	10	20	7.00	9.00	0.190	0.300	0.22	14.5613	17.5514	—	0.9416
508	10	10	7.00	9.00	0.300	0.190	0.22	0.7100	4.0000	16.3073	13.0000
503	10	10	7.00	9.00	0.300	0.190	0.22	13.3106	16.0025	—	0.8727
522	10	10	7.00	9.00	0.300	0.190	0.20	0.8723	4.7000	26.4378	22.0011
503	10	10	7.00	9.00	0.300	0.190	0.20	23.0050	26.0034	—	1.1513

TABLE 4. SUMMARY OF RESULTS, REINFORCED CONCRETE BASEMENT SHELTERS (continued)

CASE	a (ft)	b (ft)	d (in)	t (in)	As1 sqin/ft	As2 sqin/ft	Sf	Probability of Failure (Upper bound, Lower bound)			
								10%	50%	90%	Survival
								10%	50%	90%	Upper bound, Lower bound
702	12	24	5.22	6.25	0.130	0.260	0.13	0.4485	2.8377	4.9248	6.8892
703	12	24	5.22	6.25	0.130	0.260	0.13	5.0061	6.2255	8.5015	4.5851
702	12	12	5.22	6.25	0.130	0.260	0.21	0.7947	4.4314	7.6853	8.9186
703	12	12	5.22	6.25	0.130	0.260	0.21	7.8208	13.5691	8.9574	7.9616
702	6	12	5.22	6.25	0.130	0.260	0.36	0.9236	5.5328	21.1266	23.7738
703	6	12	5.22	6.25	0.130	0.260	0.36	21.6098	26.2308	1.3294	26.7975
702	6	12	5.22	6.25	0.260	0.260	0.36	0.8271	4.4943	18.6070	20.5916
703	6	12	5.22	6.25	0.260	0.260	0.36	19.0301	---	1.1922	19.1175
702	6	6	5.22	6.25	0.260	0.260	0.47	1.0327	6.1555	36.0448	36.5448
703	6	6	5.22	6.25	0.260	0.260	0.47	33.4569	---	1.7534	36.9628
802	16	32	7.80	9.00	0.200	0.350	0.10	0.5249	3.6828	5.9284	7.5853
803	16	32	7.80	9.00	0.200	0.350	0.10	5.9980	7.8699	8.5753	4.8523
802	16	16	7.80	9.00	0.200	0.350	0.16	0.7998	5.0694	9.3681	11.2864
803	16	16	7.80	9.00	0.200	0.350	0.16	9.5162	11.9420	8.9192	9.0461
902	8	16	7.80	9.00	0.200	0.350	0.28	0.9940	4.9828	25.3834	28.9557
903	8	16	7.80	9.00	0.200	0.350	0.28	25.8705	36.4408	1.1892	10.7362
902	8	8	7.80	9.00	0.350	0.350	0.28	0.8239	4.8167	22.8860	26.0407
903	8	8	7.80	9.00	0.350	0.350	0.28	23.3280	27.3791	1.8738	9.7585
902	8	8	7.80	9.00	0.350	0.350	0.37	1.0288	6.1452	44.0470	43.3911
903	8	8	7.80	9.00	0.350	0.350	0.37	39.9911	45.6074	1.5104	38.3104
902	40	40	10.49	11.50	0.250	0.470	0.08	0.6171	3.8485	11.5170	9.2825
903	40	40	10.49	11.50	0.250	0.470	0.08	6.9519	9.7145	8.6881	4.7196
902	20	20	10.49	11.50	0.250	0.470	0.12	0.8916	4.9220	15.5914	13.4895
903	20	20	10.49	11.50	0.250	0.470	0.12	10.9322	14.1748	8.8962	9.1408
902	10	20	10.49	11.50	0.250	0.470	0.22	0.9958	5.6123	29.3895	34.4968
903	10	20	10.49	11.50	0.250	0.470	0.22	29.8769	36.0953	1.1084	9.1034
902	10	10	10.49	11.50	0.470	0.470	0.22	0.8150	5.0446	26.1059	30.3874
903	10	10	10.49	11.50	0.470	0.470	0.22	26.6348	36.0910	8.9933	8.1879
902	10	10	10.49	11.50	0.470	0.470	0.29	1.0311	8.1983	45.8253	50.8877
903	10	10	10.49	11.50	0.470	0.470	0.29	44.4269	51.4442	1.3680	19.0080

TABLE 4. SUMMARY OF RESULTS, REINFORCED CONCRETE BASEMENT SHELTERS (concluded)

CASE	a (ft.)	b (ft.)	d (in.)	h (in.)	q _{al} (sqin/ft)	q _{u3} (sqin/ft)	sf	Prob. of People Survival	50%	90%	Upper bound, Lower bound	50%	90%
1002	12	20	8.74	7.75	0.180	0.350	0.13	0.0036	3.4848	11.4409	8.7001	10.4053	---
1003	12	20	8.74	7.75	0.160	0.350	0.13	0.0179	10.8587	---	0.6763	4.9915	---
1008	12	12	8.74	7.75	0.160	0.350	0.21	0.0002	4.9054	16.4330	13.4529	15.5109	---
1003	12	12	8.74	7.75	0.160	0.350	0.21	13.5365	16.1372	---	1.0742	12.0641	---
1022	6	12	8.74	7.75	0.160	0.350	0.36	1.0052	5.4515	39.7587	36.5712	40.5148	---
1003	6	12	8.74	7.75	0.160	0.350	0.36	37.8045	42.6441	---	1.4495	16.5567	---
1002	6	12	8.74	7.75	0.350	0.160	0.36	0.8766	4.9079	35.1971	31.8302	35.1637	---
1003	6	12	8.74	7.75	0.350	0.160	0.36	32.5211	37.3650	---	1.2799	26.8906	---
1022	6	6	8.74	7.75	0.350	0.160	0.47	1.1877	6.7693	---	55.7205	---	---
1003	6	6	8.74	7.75	0.350	0.160	0.47	48.3839	---	---	1.9410	51.3136	---
1102	16	30	9.72	10.75	0.230	0.470	0.10	0.7065	5.7264	14.3176	9.8906	12.7277	---
1103	16	30	9.72	10.75	0.230	0.470	0.10	10.0168	13.2310	---	0.7752	5.8617	---
1102	16	16	9.72	10.75	0.230	0.470	0.16	0.8706	5.5581	20.0607	15.3740	18.5543	---
1103	16	16	9.72	10.75	0.230	0.470	0.16	15.5870	19.2361	---	1.0081	8.9144	---
1102	8	16	9.72	10.75	0.230	0.470	0.28	0.9827	5.3994	45.7008	41.5028	46.8328	---
1103	8	16	9.72	10.75	0.230	0.470	0.28	41.3990	48.4675	---	1.2879	11.6716	---
1102	8	16	9.72	10.75	0.470	0.230	0.28	0.8650	4.8698	41.1565	36.5474	41.4161	---
1103	8	16	9.72	10.75	0.470	0.230	0.28	37.6675	43.0232	---	1.1420	10.3650	---
1122	8	8	9.72	10.75	0.470	0.230	0.37	1.1189	6.7324	---	62.9695	---	---
1103	8	8	9.72	10.75	0.470	0.230	0.37	48.3846	---	---	1.6431	40.6793	---
1202	20	40	12.83	14.00	0.310	0.620	0.08	0.8146	6.0576	18.4669	11.3657	15.4906	---
1203	20	40	12.83	14.00	0.310	0.620	0.08	11.4884	16.3337	---	0.8784	6.2208	26.2212
1202	20	20	12.83	14.00	0.310	0.620	0.12	0.8962	5.4984	24.8034	17.6726	22.1846	---
1203	20	20	12.83	14.00	0.310	0.620	0.12	17.8937	23.2184	---	0.9967	7.4906	---
1202	10	20	12.83	14.00	0.310	0.620	0.22	0.9985	5.9336	52.6847	47.4017	54.8741	---
1203	10	20	12.83	14.00	0.310	0.620	0.22	45.3765	54.6988	---	1.2254	10.0210	---
1202	10	20	12.83	14.00	0.620	0.310	0.22	0.8811	5.4810	47.6168	41.8775	48.4370	---
1203	10	20	12.83	14.00	0.620	0.310	0.22	41.6042	49.5822	---	1.0873	8.9242	---
1202	10	10	12.83	14.00	0.620	0.310	0.29	1.1490	6.8878	---	---	---	---
1203	10	10	12.83	14.00	0.620	0.310	0.29	48.3846	---	---	1.5125	14.0026	---

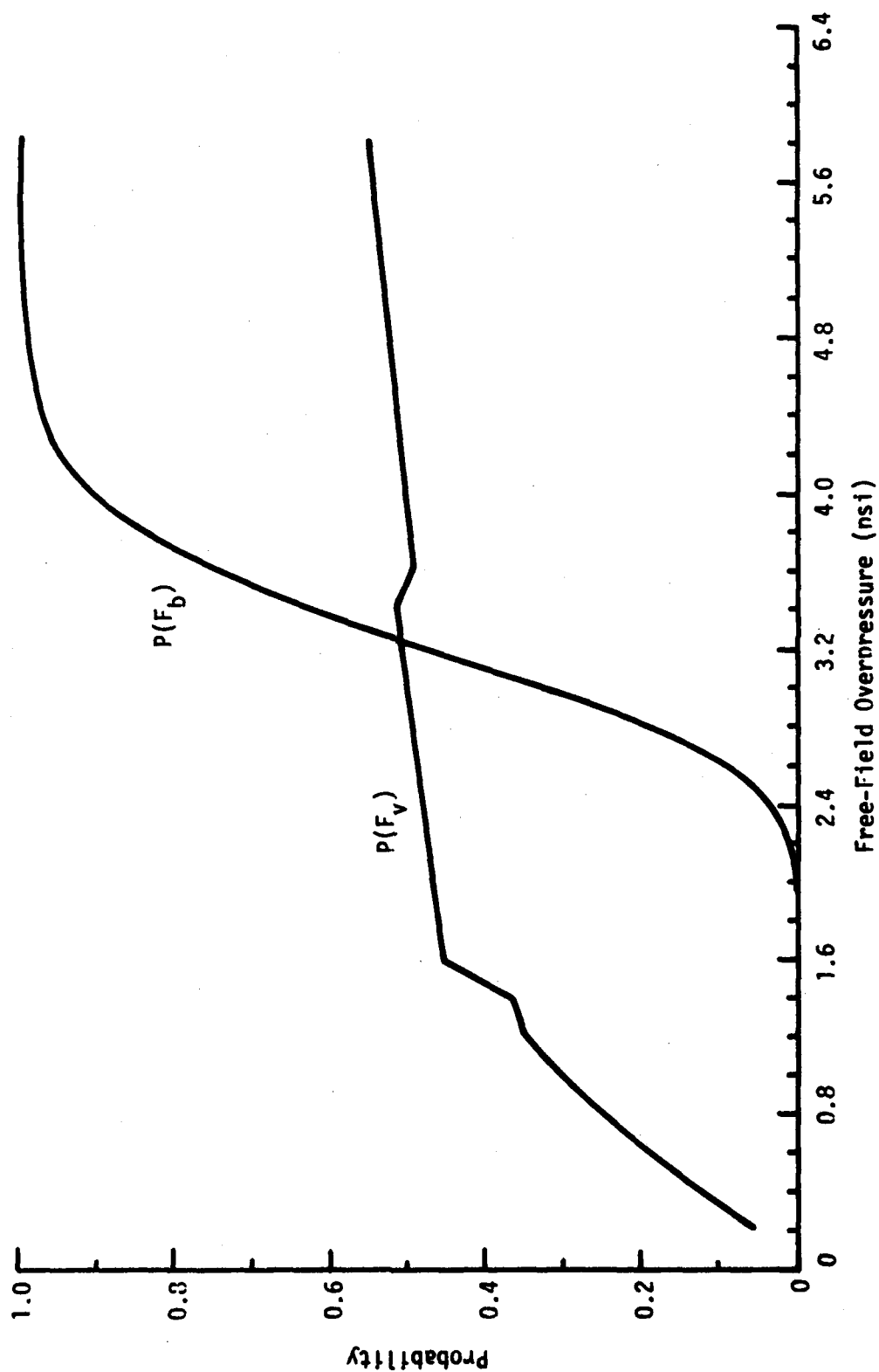


Figure 10. Failure probabilities due to flexure and shear, Case 1A.

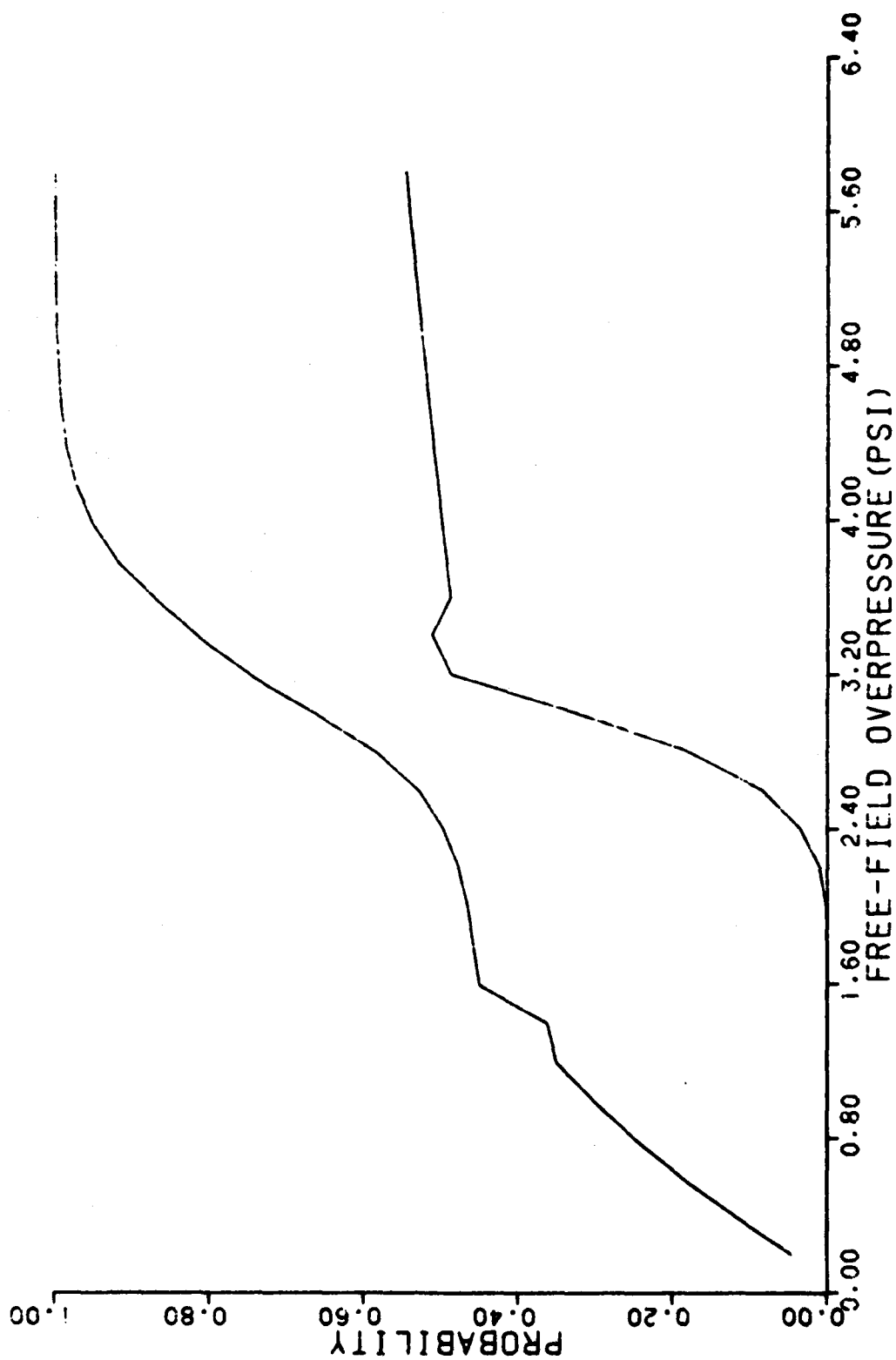


Figure 11. Probability of slab failure (upper and lower bounds) Case 1A.

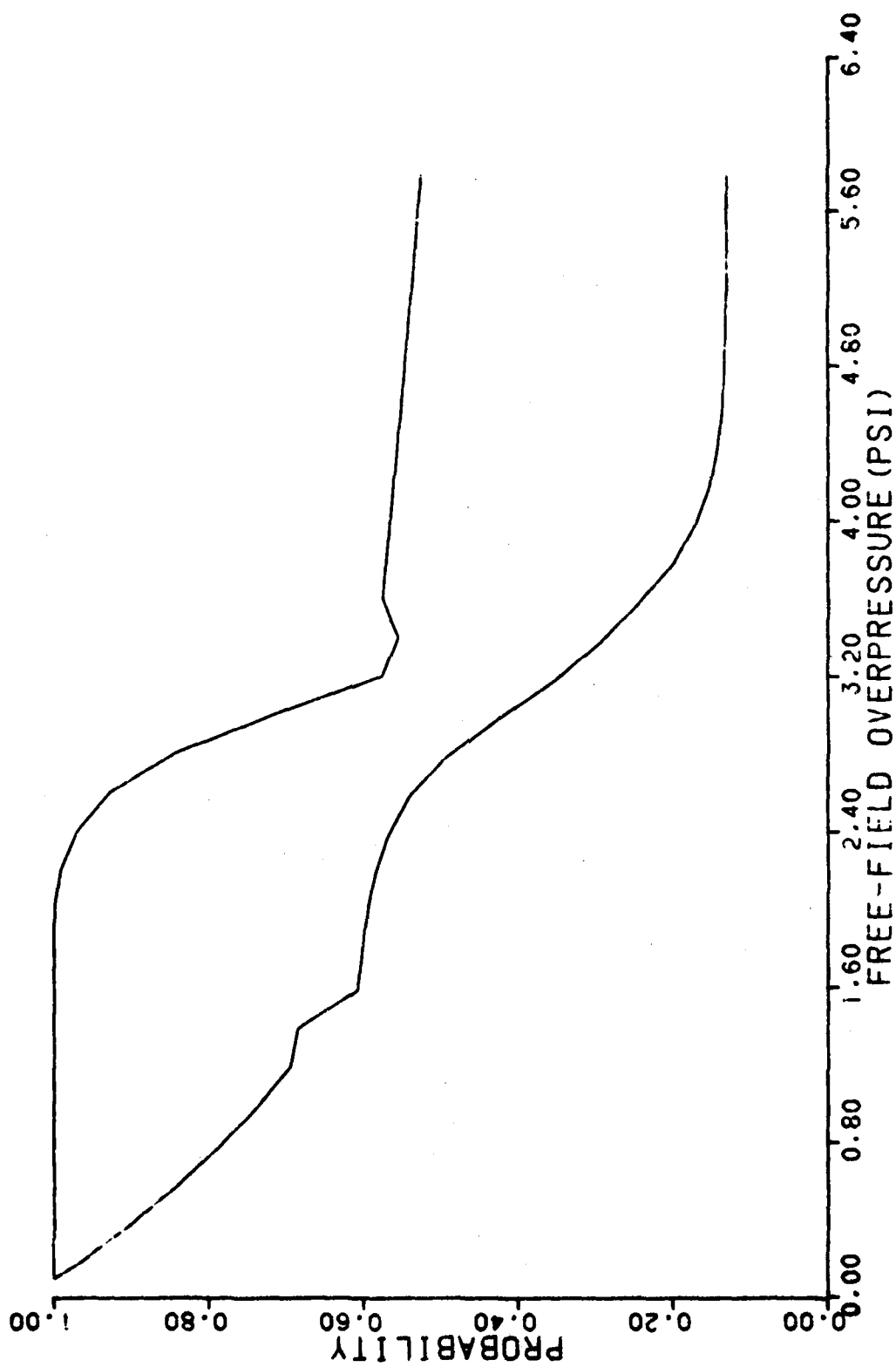


Figure 12. Probability of people survival (upper and lower bounds) Case 1A.

TABLE 5. OVERPRESSURE RANGES AT THE 90%
AND 50% PROBABILITY OF SURVIVAL (CASE 1)

Probability Level	Expedient Upgrading Scheme				
	A	B	C	D	E
90%	0.39-2.64	0.55-4.18	0.87-11.73	0.86-10.42	1.58-18.20
50%	2.77-6.60	4.57-12.90	12.47-26.10	11.21-	19.96-

For schemes D and E the upper bonds are not included at the 50% probability of survival level. The reason is that the curves are very flat after a certain point, and do not intersect the particular probability value in the overpressure range of interest (see Figure 12, for example). This is also the reason why numbers are missing in certain columns in Table 4. It is evident from these results that expedient upgrading can be very effective in providing protection. The 50% survival probability at 2.77 psi for the basic slab becomes almost 20 psi when upgrading scheme E is employed. Similar trends will be noted for the other slabs.

If it is a matter of choosing between two basements for expedient upgrading, then obviously the one that was designed for a higher live load should be chosen assuming that both are in good physical condition and the design loads are known for each. The key item in expedient upgrading is the correct design of the supporting system and its correct implementation.

It is recommended that experimental studies be initiated whose objective would be to generate experimental data on the response of reinforced concrete slabs subjected to dynamic loadings. We need experimental data in the response range approaching failure. Available experimental data on the shear failure of slabs, the distribution of reactive forces along the supports, and the interaction of flexure and shear prior to and at the point of failure, is especially limited at this time. Reliable data would aid in the development of accurate failure theories and also in the development of design and implementation criteria for expedient upgrading schemes.

Since the upper and lower bounds on the failure probability are fairly far apart for most cases studied (see Figure 12), it becomes useful to

determine the correlation that exists between the two failure modes and then to determine the actual failure probability. This was not done in this study because the methodology for doing this was not available. For the present, it is believed that the lower bound should be used as a conservative measure of the probability of people survival.

6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY

The study reported here was concerned with predicting the probability of survival of people located in expediently upgraded, conventional basements when subjected to the blast effects produced by the detonation of a 1-MT weapon near the ground surface. Two categories of basements are considered, i.e., basements of engineered buildings and basements of single family residences.

The first category includes basements of low-rise engineered buildings. The basement walls are unexposed and the first floor slab (the slab over the basement) is at grade. The first floor slab is the primary structural component for the basement as far as protection from the blast is concerned. Its collapse will result in casualties due to debris impact and blast winds and pressures entering shelter areas when the shelter envelope is breached.

The slab over the basement was designed as a one-way reinforced concrete system for live loads of 50, 80, 125, and 250 psf and span lengths of 12, 16, and 20 ft. This includes a total of twelve conventional basements having a representative range of use classes. Each of the twelve basements was analyzed as expediently upgraded using four different upgrading schemes. Upgrading is accomplished by providing supports that reduce the effective span of the slab and by blocking off all openings into the basement. This resulted in sixty shelters of different strengths which include the conventional, unupgraded slab as the base case.

The second category includes four conventional wood frame residences with full basements. These basements were also analyzed as expediently upgraded. Again, upgrading consisted of providing intermediate supports for the joist floor system, blocking off all openings into the basement, and mounding the building with soil up to the first floor level, about 2 to 3 ft from grade. Expedient upgrading included "studwall" and "post and beam" concepts. Six shelters were analyzed. First, each basement was analyzed

as upgraded using the studwall scheme; second, two of the basements were reanalyzed using the "post and beam" concept.

A probability of survival function was developed for each shelter and each upgrading scheme. The procedure used to accomplish this consists of two parts. The first is a probabilistic structural analysis which determines the probability of failure to the shelter envelope. The second part is a probabilistic people survival analysis which considers two casualty producing mechanisms, i.e., debris impact and associated effects from the collapse of the primary structural system and primary blast. The probability of structural failure is made use of in computing the probability of survival against debris effects.

The analysis method briefly described above was formulated in the course of this study and the previous effort (Ref 5), and a portion of it was computerized. The computer program is capable of analyzing reinforced concrete shelters of the type discussed previously in this chapter, and of predicting the probability of survival for shelter occupants against the effects of blast.

6.2 CONCLUSIONS

On the basis of the study conducted and results obtained, the following conclusions are made:

- (1) A method for computing the probability of failure of structures (Ref 15) was examined and applied to the analysis of personnel shelters. This method is capable of considering all structural components making up the structure and the respective failure modes of each component. This method is the most rational that is available at this time and the results are believed to be the most reliable of those produced in this subject area to date.
- (2) Expedient upgrading can be very effective in increasing the live saving potential of conventional reinforced concrete basements. Conventional basements with one-way reinforced concrete overhead floor systems designed for a live load of 50 psf can be expediently upgraded to result in a probability of survival of 50% at 20 psi. Slabs originally designed for a higher live load, 125 psf for example, can be easily upgraded to result in a probability of survival of 50% beyond 30 psi. It can, therefore, be supposed that two-way floor

systems, which are generally stronger than one-way floor system, have the capacity of providing even greater protection when effectively upgraded.

- (3) Conventional wood framed basements are capable of being expediently upgraded to provide protection beyond 5 psi at a probability of survival of 50% or better.
- (4) The analysis procedure presented here is sufficiently general and can be readily extended to include the influence of other hazards which accompany a nuclear weapon attack, i.e., prompt nuclear radiation, fires, and fallout radiation.
- (5) A capability should be formulated and included in the analysis to study the effects of evasive action taken by shelter occupants on the probability of survival.

6.3 RECOMMENDATIONS

The previous effort (Ref 5) and the study reported here have been very useful in formulating the people survivability problem on a rational probabilistic basis. The approach is very promising and if allowed to develop further will produce a reliable computational tool for the rating of shelter spaces, evaluating alternative shelter systems, and for performing damage limiting studies. With this end in mind, two tasks are recommended.

6.3.1 Experimental Task

There is a need for experimental studies to be initiated to generate data for a better understanding of how reinforced concrete structural components respond in the range approaching failure, i.e., what failure modes are introduced, how they interact with each other for different slabs, what is the influence of boundary conditions and loadings on the modes of failure. Experimental studies should be conducted to generate data capable of improving the current formulations of the following failure criteria:

- Failure criteria for horizontally oriented reinforced concrete slabs; one-way and two-way floor systems
- Failure criteria for vertically oriented slabs in contact with the soil; basement walls
- Failure criteria for columns and beams.

A test plan, outlining the number of experiments that would be needed to produce the necessary data cannot be produced at this time. The first task would be a review of all available experimental data on this subject. With this task completed it would be possible to outline a preliminary test plan.

6.3.2 Analytic Task

The computer program developed thus far must be further developed to include the capability to analyze the following structural systems and to include related aspects:

(a) Individual Structural Components

In addition to the library of structural components included in the computer program at this time, the following should be implemented:

- Flat slabs
- Flat plates
- One-way slabs
- Beams (steel, reinforced concrete)
- Columns (steel, reinforced concrete)
- Composite steel and concrete systems
- Masonry systems.

(b) Weapon Effects Hazards

In addition to blast and debris effects included in the computer program at this time, other nuclear weapon effects and indirect hazards should be included.

- Prompt nuclear radiation
- Fallout radiation
- Ground shock
- Fires.

(c) Casualty Data

Available data for estimating the level of casualty experienced by individuals against the various hazards should be reviewed with the object of making the current casualty predicting process more reliable.

All of the aspects outlined would be considered within the probabilistic framework outlined in Section 3.

APPENDIX A

PROBABILITY OF PEOPLE SURVIVAL IN THE BASEMENT OF A WOOD FRAME RESIDENCE

Procedure used in determining the probability of people survival in basements of single-family, wood frame residences is presented in this appendix. The particular building analyzed is the "Dunes" house (Ref 20). This is a one-story, single-family frame residence with a full basement. The floor system over the basement is approximately 1 ft above grade.

The analysis described is for a basement expediently upgraded by providing stud walls in the basement as additional supports for the floor system, blocking windows and doors, and mounding the structure on the outside up to the first floor level.

Two cases are considered, i.e., with and without soil cover (1 ft depth) over the floor surface for radiation protection.

A.1 FAILURE PROBABILITY OF THE WOOD FLOOR SYSTEM

This section presents calculations leading to the determination of the probability of failure of the expediently upgraded floor system. This floor system consists of joists, girder, columns, and stud walls.

A.1.1 Material Properties

The entire floor system, Figures A-1 and A-2, consists of Jack Pine whose properties, for several loading conditions, are given in Table A-1. Specific properties used in this analysis are for 1 sec load duration. The coefficient of variation associated with each of these values is taken as 0.20.

A.1.2 Applied Load

Two load cases are considered in the analysis, i.e.,

- (1) A uniformly distributed mean pressure, \bar{p} (psi), with a coefficient of variation, $\Omega_p = 0.20$.
- (2) A uniformly distributed mean pressure, \bar{p} , plus 1 ft of soil load.

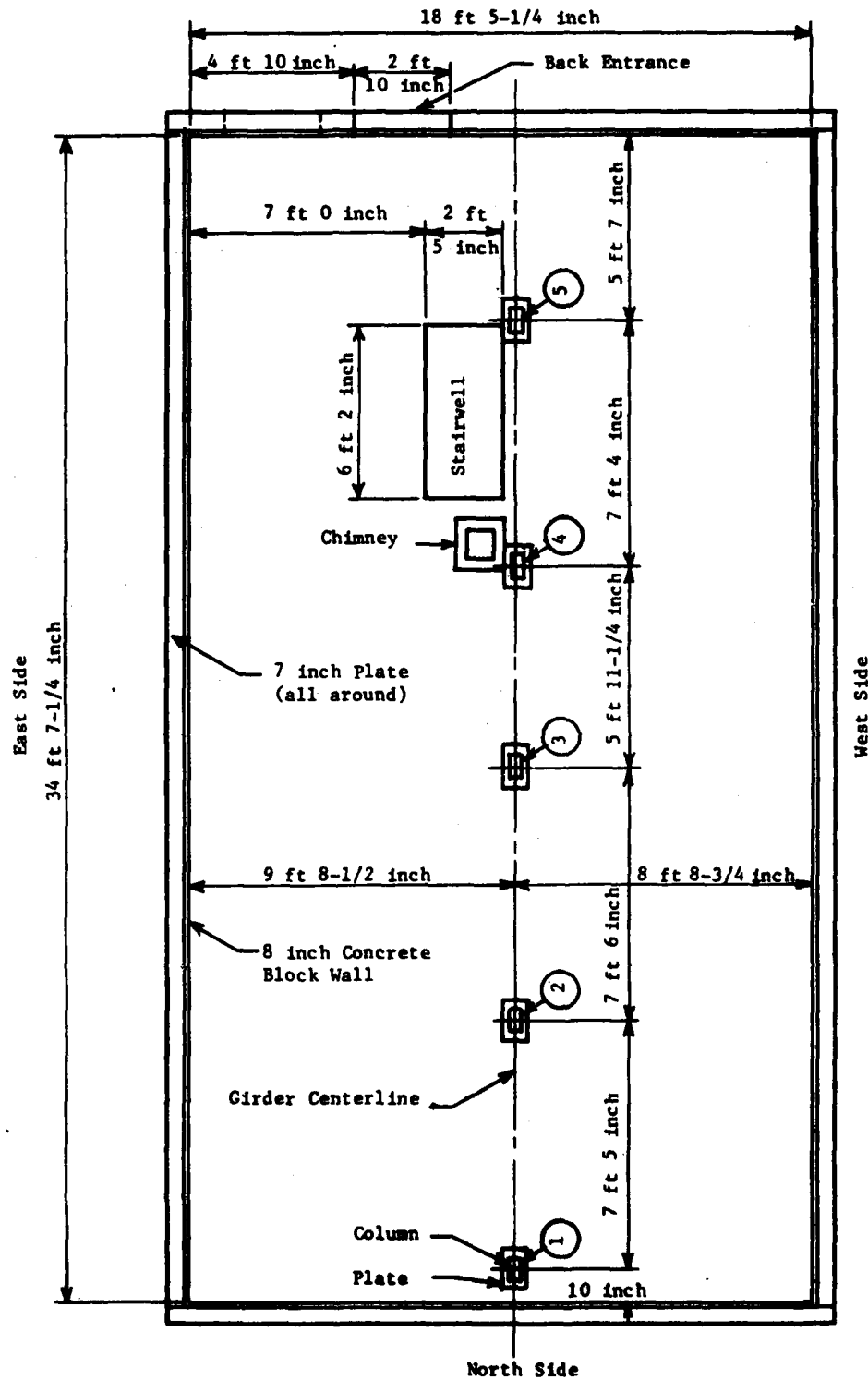


Figure A-1. Basement plan.
(Dunes house)

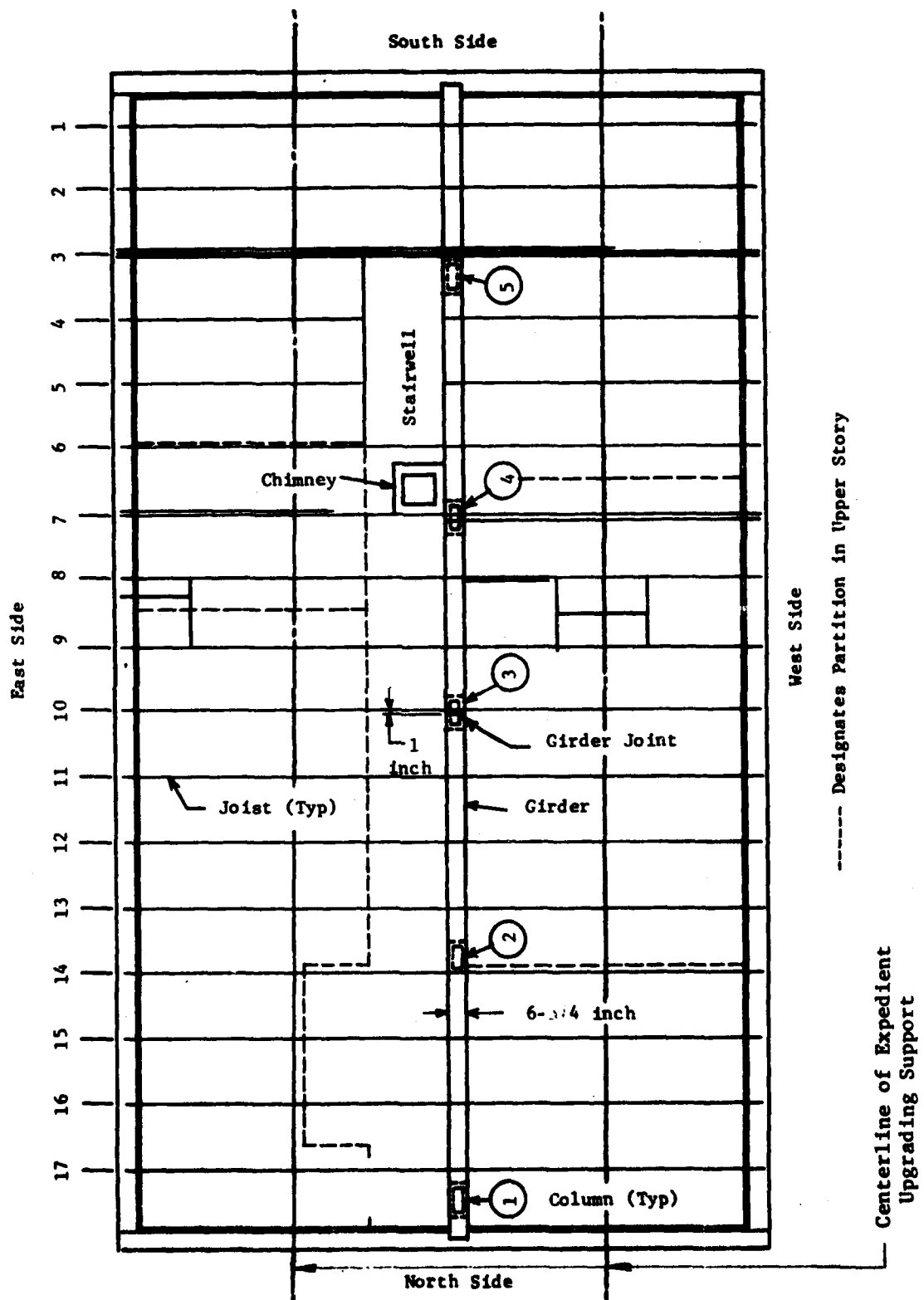


Figure A-2. Joist, girder, and upper story partition layout.
(Dunes house)

TABLE A-1. MECHANICAL PROPERTIES OF JOISTS*

Property**	Mean Clear Wood Strength Value, psi	Strength Factor***	Normal Duration of Load Factor	Size Factor	Combined Factor	Normal Duration Mean Strength Value, psi	1.25 day Duration Mean Strength Value, psi	1 hr Duration Mean Strength Value, psi	1 sec Duration Mean Strength Value, psi
F_b	9,900	0.63	1/1.6	0.89	0.35	3,450	4,600	5,150	7,100
F_c	5,660	0.78	2/3	----	0.52	2,950	3,900	4,350	6,050
F_v	1,170	0.50	1/1.6	----	0.31	365	480	715	750
F_t	9,900	0.37	1/1.6	----	0.23	2,300	3,000	4,450	4,700
F_{cl}	580	1.00	1/1.1	----	0.91	525	525	525	525
$E/1000$	1,350	1.00	1	----	1.00	1,350	1,350	1,350	1,350

* Jack Pine

** F_b = Rupture Strength

F_c = Compression Parallel to Grain

F_v = Shear Parallel to Grain

F_t = Tension Parallel to Grain

F_{cl} = Compression Perpendicular to Grain

E = Modulus of Elasticity

*** Structural Light Framing, Select Structural (Table 8, Ref 21)

The dead load of the floor system is neglected. The uniformly distributed pressure, \bar{p} , is assumed to be applied for 1 sec.

A.1.3 Member Sizes

Joists: 1.625 in. by 5.625 in. with an average spacing of 24.12 in.
 Girder: 5.5 in. by 6.75 in.
 Columns: 4.0 in. by 8.0 in.
 Studwalls: Columns 2 in. by 4 in. with bracing at mid-height (Figure A-3).

A.1.4 Assumptions

- (1) Joists 1 through 17 (Figure A-2) are identical and continuous over the girder and the stud walls.
- (2) The connections between the flooring and the joists are not sufficiently strong to develop composite action. Therefore, the joists act independent of the flooring.
- (3) The flexibility of the girder in calculating joist stresses is neglected. The girder is assumed to provide a rigid support for the joists.
- (4) Resistance along a member is perfectly correlated, i.e., failure of the member occurs at the point of maximum load effect.

A.1.5 Failure Probability of a Joist

The joist loading, shear and bending moment diagrams are shown in Figure A-4.

$$M_{\max} = M = 9759 \bar{p} \text{ lb/in. } (\bar{p} \text{ is in lb/in.}^2)$$

$$V_{\max} = V = 891 \bar{p} \text{ lb } (\bar{p} \text{ is in lb/in.}^2)$$

$$\Omega_M = \Omega_V = 0.20.$$

A.1.5.1 Modes of Failure - Bending

$$\phi_b = N_{g1} \frac{F_b}{F_b} = N_{g1} F_b S/M \quad (A-1)$$

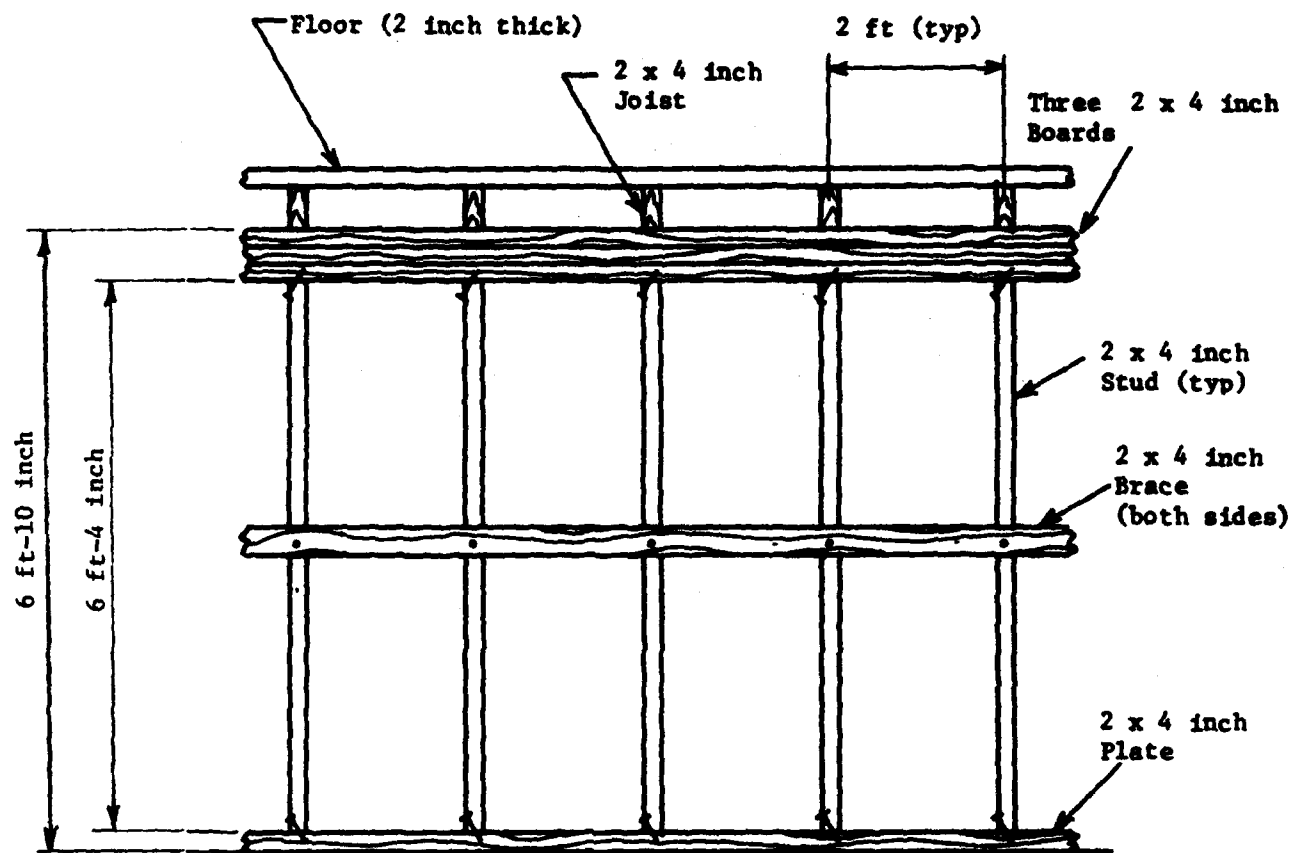


Figure A-3. Expedient upgrading.

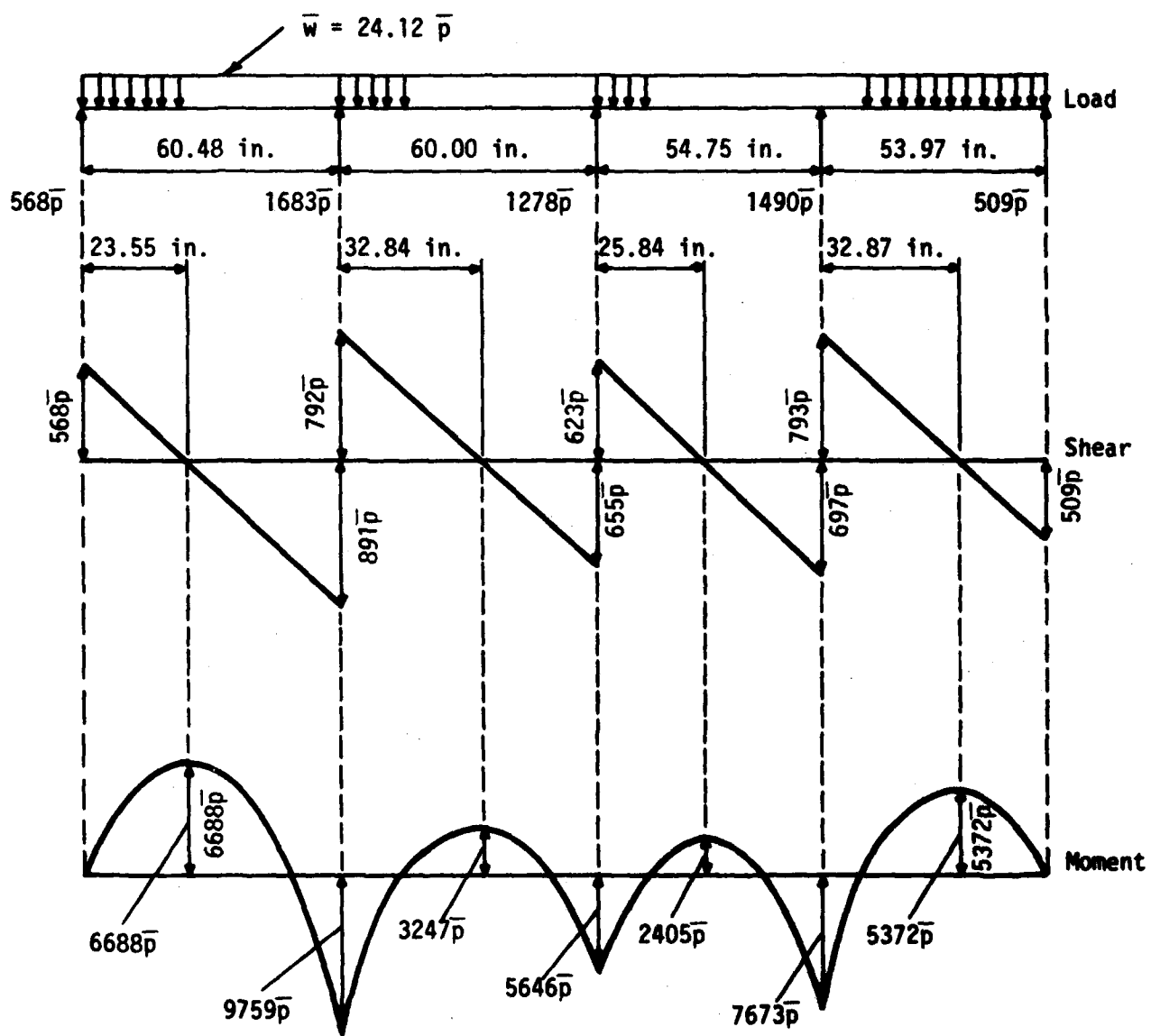


Figure A-4. Joist loading, shear and bending moment diagrams.

where θ_b = safety factor in bending
 N_{g1} = correction factor on the flexural formula
 F_b = rupture strength (see Table A-1)
 f_b = applied bending stress
 S = section modulus.

$$\bar{\theta}_b = \bar{N}_{g1} \bar{F}_b \bar{S}/\bar{M} \quad (A-2)$$

$\bar{\theta}_b$ = mean safety factor and the remaining parameters are mean values of those identified in equation (A-1).

$$\Omega_{\theta_b} = \sqrt{\Omega_{g1}^2 + \Omega_b^2 + \Omega_S^2 + \Omega_M^2} \quad (A-3)$$

where Ω_{θ_b} , Ω_{g1} , Ω_b , Ω_S , and Ω_M are coefficients of variation of $\bar{\theta}_b$, \bar{N}_{g1} , \bar{F}_b , \bar{S} , and \bar{M} , respectively.

a. Uncertainty in the Flexure Formula

Due to the difference between the idealized linear elastic formula and the real case, assume $\bar{N}_{g1} = 0.95$ with a uniform distribution between 0.90 and 1.0, as shown in Figure A-5 below.

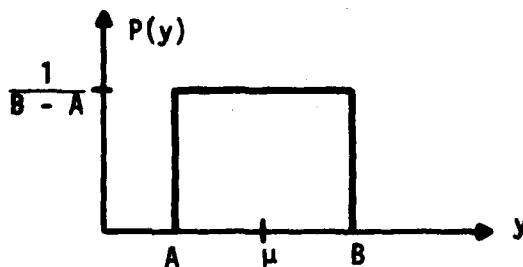


Figure A-5. Uniform distribution.

For a uniform distribution, see Figure A-5.

$$\Omega = \frac{1}{\sqrt{3}} \frac{B - A}{B + A} \quad (\text{Ref 22}) \quad (\text{A-4})$$

$$\text{thus, } \Omega_{g1} = \frac{1}{\sqrt{3}} \frac{0.10}{1.90} = 0.03.$$

b. Uncertainty in the Section Modulus, S

$$S = \frac{\bar{b}\bar{h}^2}{b} \quad (\text{A-5})$$

where \bar{b} = width of the joist
 \bar{h} = depth of the joist.

If b and h are perfectly correlated, then the coefficient variation of S is

$$\Omega_S = \sqrt{\Omega_b^2 + 4\Omega_h^2 + 4\Omega_b\Omega_h} \quad (\text{A-6})$$

Assume that $\delta_b = \delta_h = 0.05$. Also, due to humidity effects, let $\Delta_b = \Delta_h = 0.05$. Thus

$$\Omega_h = \Omega_b = \sqrt{\delta_b^2 + \Delta_b^2} = 0.07$$

$$\Omega_S = \sqrt{(0.07)^2 + 4(0.07)^2 + 4(0.07)(0.07)} = 0.21$$

Due to possible existence of notches, the mean height of the joists is taken as 0.8h. The section modulus, \bar{S} , becomes

$$\bar{S} = \frac{1.625(0.8 \times 5.625)^2}{b} = 5.484 \text{ (in.)}^3$$

c. Determination of $\bar{\theta}_b$ and Ω_{θ_b}

From equations (A-1) and (A-3),

$$\bar{\theta}_b = 0.95(7100)(5.484)/9759 \bar{p} = 3.790/\bar{p}$$

$$\Omega_{\theta_b} = [(0.03)^2 + (0.20)^2 + (0.21)^2 + (0.20)^2]^{1/2} = 0.354$$

d. Probability of Failure Due to Bending

$$P(F_b) = 1 - \Phi \left[\frac{\ln(3.79/\bar{p})}{0.354} \right] = 1 - \Phi \left[\frac{1.332 - \ln \bar{p}}{0.354} \right] \quad (A-7)$$

A.1.5.2 Modes of Failure - Shear

$$\theta_v = N_{g2} \frac{F_v}{f_v} = 2N_{g2} F_v A/3V \quad (A-8)$$

where θ_v = safety factor in shear

N_{g2} = correction factor on the shear formula

F_v = shear strength (see Table A-1)

f_v = shear stress due to applied load

A = cross-sectional area of joist

$$\Omega_{\theta_v} = (\Omega_{g2}^2 + \Omega_{F_v}^2 + \Omega_A^2 + \Omega_v^2)^{1/2} \quad (A-9)$$

a. Uncertainties in the Shear Formula

Assume that $\Omega_{g2} = \Omega_{g1}$. Thus, $\Omega_{g2} = 0.03$

$$\Omega_{F_v} = 0.20 \text{ (estimated)}$$

$$\Omega_A = (\Omega_b^2 + \Omega_h^2 + 2\Omega_b\Omega_h)^{1/2}$$

$$\Omega_A = [(0.07)^2 + (0.07)^2 + 2(0.07)(0.07)]^{1/2} = 0.14$$

$$\Omega_V = 0.20 \text{ (estimated)}$$

b. Determination of $\bar{\theta}_V$ and Ω_{θ_V}

$$\bar{\theta}_V = 2N_{g2} F_V A/3V = 2(0.95)(750)(1.625)(0.8)(5.625)/(3)(891)\bar{p}$$

$$\bar{\theta}_V = 3.898/\bar{p}$$

$$\Omega_{\theta_V} = [(0.03)^2 + (0.20)^2 + (0.14)^2 + (0.20)^2]^{1/2} = 0.317$$

c. Probability of Failure Due to Shear

$$P(F_V) = 1 - \Phi \left[\frac{\ln(3.898/\bar{p})}{0.317} \right] = 1 - \Phi \left[\frac{1.361 - \ln \bar{p}}{0.317} \right] \quad (A-10)$$

A.1.5.3 Joist Failure Probability

The joist can fail in flexure or in shear. As discussed in Section 3.3, if the two failure modes are independent of each other then the failure probability of the joist, $P(F_j)$ is

$$P(F_j) = 1 - [1 - P(F_b)][1 - P(F_v)] \quad (A-11)$$

If the failure modes are highly correlated, then

$$P(F_j) = \max[P(F_b), P(F_v)] \quad (A-12)$$

The actual failure probability for the joist is between these two probabilities, thus

$$\max[P(F_b), P(F_v)] \leq P(F_j) \leq 1 - [1 - P(F_b)][1 - P(F_v)] \quad (A-13)$$

Failure probabilities of the joist were computed and are shown in Figure A-6. Failure probabilities $P(F_D)$ and $P(F_V)$ were computed using equations (A-7) and A-10), respectively. The upper bound failure probability was computed using equation (A-11).

It is noted (see Figure A-6) that the two bounds, i.e., $P(F_j)$ and $P(F_V)$ are fairly close. In this analysis, $P(F_j)$ is taken as the failure probability for the joist.

A.1.5.4 Failure Probability of the Joist System

When all joists are identical and subject to the same load distribution and intensity, then conditions between the joists are perfectly correlated. On this basis the failure probability of the joist system is represented with the failure probability of one joist. Therefore, the upper bound values given in Figure A-6 are conservatively considered as the failure probability of the entire joist system.

A.1.6 Failure Probability of the Girder

As shown in Figure A-2, the girder consists of two parts, i.e., the part between columns 1 and 3 (part 1) and the part between columns 3 and the south wall (part 2). The two parts are analyzed separately.

A.1.6.1 Analysis of Girder, Part 1

The configuration of this portion of the girder is shown in Figure A-7. The loading, P , is due to joists.

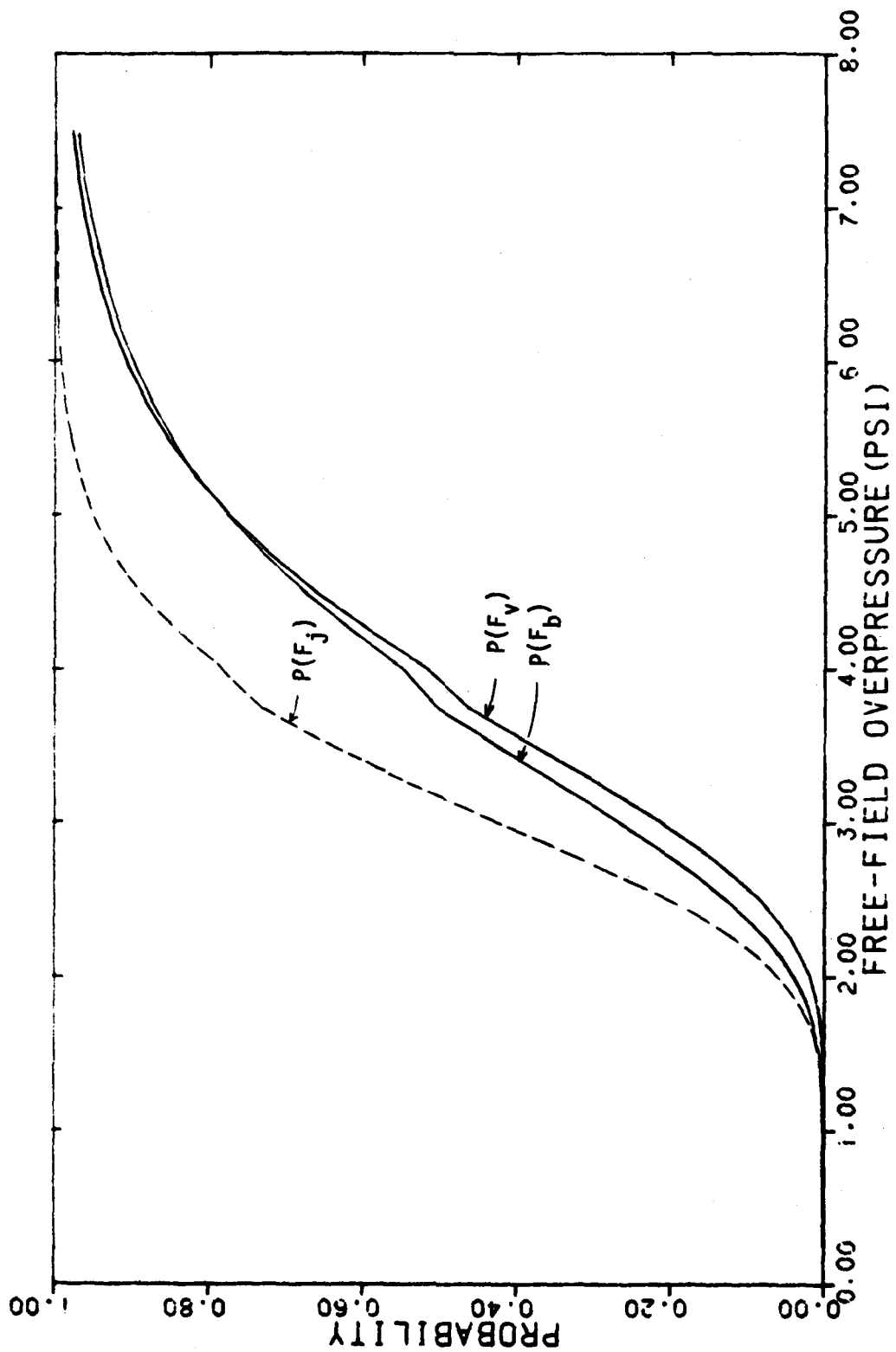


Figure A-6. Probability of joist failure.

$$\bar{P} = 1278\bar{p} \text{ (see Figure A-4)}$$

where \bar{P} = mean value of the joist load, lb

\bar{p} = mean value of the uniform pressure, psi.

$$\bar{M} = 41.323\bar{P} = 41.323(1278)\bar{p} = 52810.8\bar{p}$$

where \bar{M} = mean value of the maximum bending moment on the girder
(at the center support).

$$\bar{V} = 2.502\bar{P} = 2.502(1278)\bar{p} = 3197.6\bar{p}$$

where \bar{V} = mean value of the maximum shear on the girder
(to the left of center support).

$$\Omega_M = \Omega_V = \Omega_P = 0.20$$

a. Bending

$$\bar{\theta}_b = \bar{N}_{g1} \bar{F}_b \bar{S}/\bar{M}$$

$$\bar{S} = \frac{bh^2}{6} = \frac{6.75(5.5)^2}{6} = 34.03 \text{ (in.)}^3$$

$$\bar{\theta}_b = (0.95)(7100)(34.03)/(52810.8\bar{p}) = 4.3463/\bar{p}$$

$$\Omega_{\theta_b} = (\Omega_{g1}^2 + \Omega_{F_b}^2 + \Omega_S^2 + \Omega_M^2)^{1/2} = 0.354$$

$$P(F_b) = 1 - \Phi\left(\frac{\ln \bar{\theta}_b}{\Omega_{\theta_b}}\right) = 1 - \Phi\left(\frac{1.469 - \ln \bar{p}}{0.354}\right) \quad (\text{A-14})$$

b. Shear

$$\bar{\theta}_v = 2\bar{N}_{g2} \bar{F}_v \bar{A}/3\bar{V}$$

$$\bar{\theta}_v = 2(0.95)(750)(6.75)(5.5)/3(3197.6\bar{p}) = 5.515/\bar{p}$$

$$\Omega_{\theta_v} = 0.317 \text{ (Determined earlier in connection with joist analysis)}$$

$$P(F_v) = 1 - \Phi \left(\frac{\ln \bar{\theta}_v}{\Omega_{\theta_v}} \right) = 1 - \Phi \left(\frac{1.707 - \ln \bar{p}}{0.317} \right) \quad (A-15)$$

A.1.6.2 Analysis of Girder, Part 2

The configuration of this portion of the girder is shown in Figure A-8.

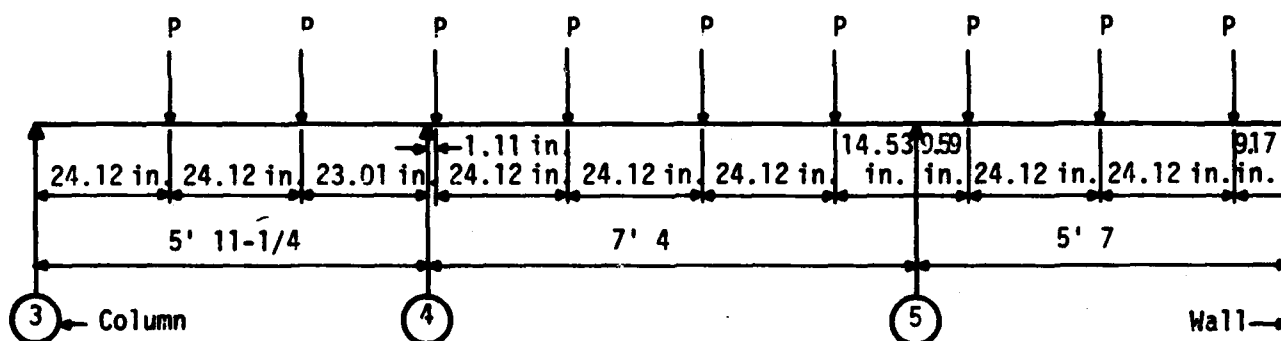


Figure A-8. Girder loading, Part 2.

$$\bar{P} = 1278\bar{p} \text{ (see Figure A-4)}$$

$$\bar{M} = 26.382\bar{P} = 26.382(1278\bar{p}) = 33716.2\bar{p}$$

where \bar{M} = mean value of the maximum moment on the girder

$$\bar{V} = 2.289\bar{P} = 2.289(1278\bar{p}) = 2925.3\bar{p}$$

where \bar{V} = mean value of the maximum shear on the girder

$$\bar{\theta}_b = K_{g1} F_b S/\bar{M}$$

$$\bar{\theta}_b = (0.95)(7100)(34.03)/33716.2\bar{p} = 6.808/\bar{p}$$

$$\Omega_{\theta_b} = 0.354 \text{ (computed previously)}$$

$$P(F_b) = 1 - \Phi \left(\frac{1.918 - \ln \bar{p}}{0.354} \right) \quad (A-16)$$

$$\bar{\theta}_v = 2\bar{N}_{g2} \bar{F}_v \bar{A}/3\bar{V}$$

$$\bar{\theta}_v = 2(0.95)(750)(6.75)(5.5)/3(2925.3\bar{p}) = 6.028/\bar{p}$$

$$\Omega_{\theta_v} = 0.317 \text{ (computed previously)}$$

$$P(F_v) = 1 - \Phi \left(\frac{1.796 - \ln \bar{p}}{0.317} \right) \quad (A-17)$$

Failure probabilities for the girder, parts 1 and 2, are shown in Figures A-9 and A-10. As previously, three curves are given, i.e., probability of failure due to bending, $P(F_b)$, probability of failure due to shear, $P(F_v)$, and the upper bound probabilities, $P(F_{g1})$ and $P(F_{g2})$, computed using equation (A-13). In each case, the actual failure probability is between the bounds of $P(F_v)$ and $P(F_{gi})$, $i = 1, 2$.

A.1.7 Failure Probability of Columns

A.1.7.1 Existing Columns

The location and spacing of columns is shown in Figures A-1 and A-2. For a uniformly distributed pressure load over the floor surface, the axial loads on the five columns have the following values:

$$P_1 = 1.498P$$

$$P_2 = 4.592P$$

$$P_3 = 2.544P$$

$$P_4 = 3.655P$$

$$P_5 = 3.595P$$

where $P = 1278p$, p = the uniformly distributed floor load in psi (lb/in.²).

The following formula (Ref 23) was used for evaluating the failure probability of the five timber columns:

$$\frac{P}{A} = F_c = \frac{0.30E}{(l/d)^2} \quad (A-18)$$

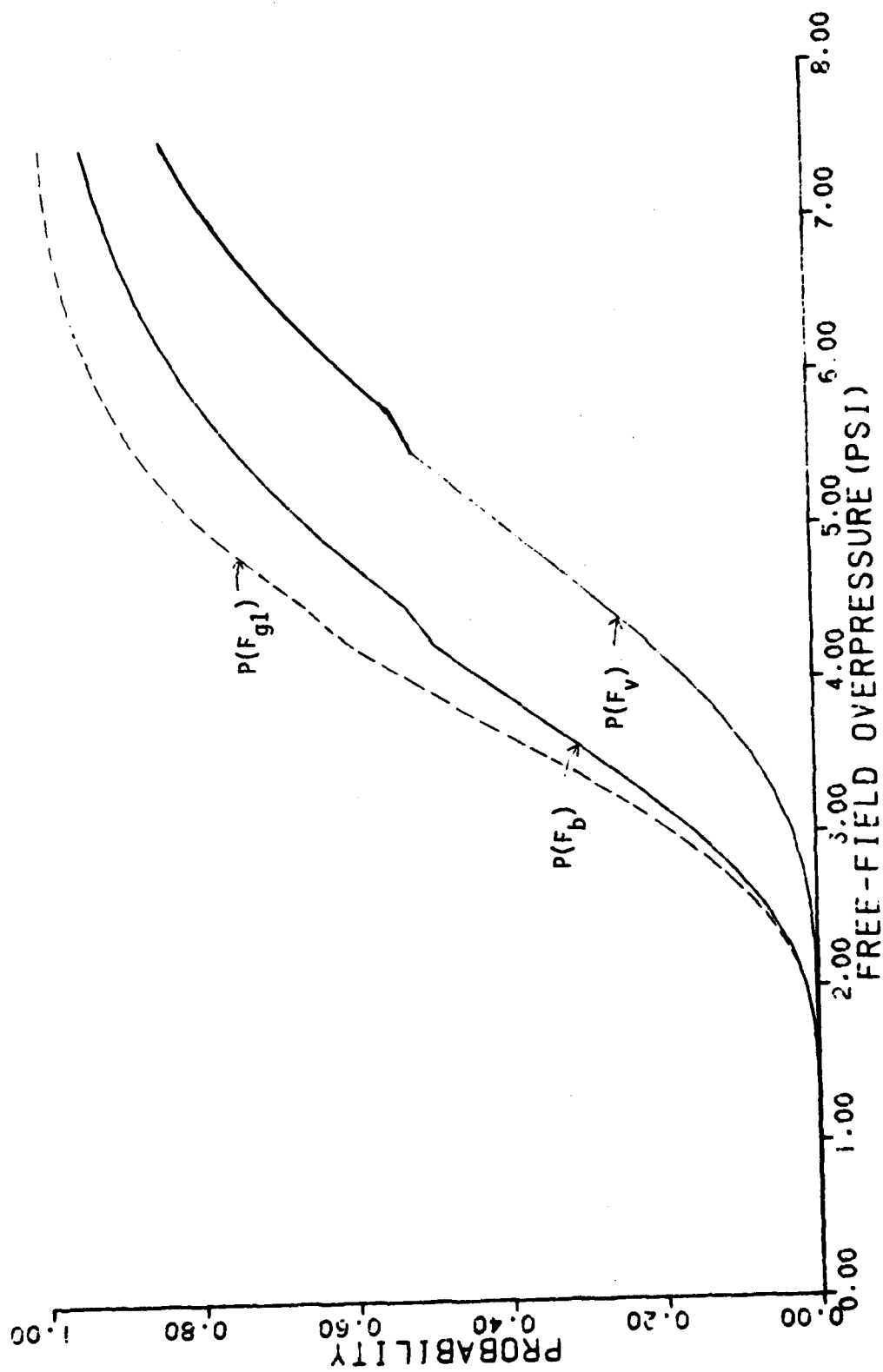


Figure A-9. Probability of girder failure (Part 1).

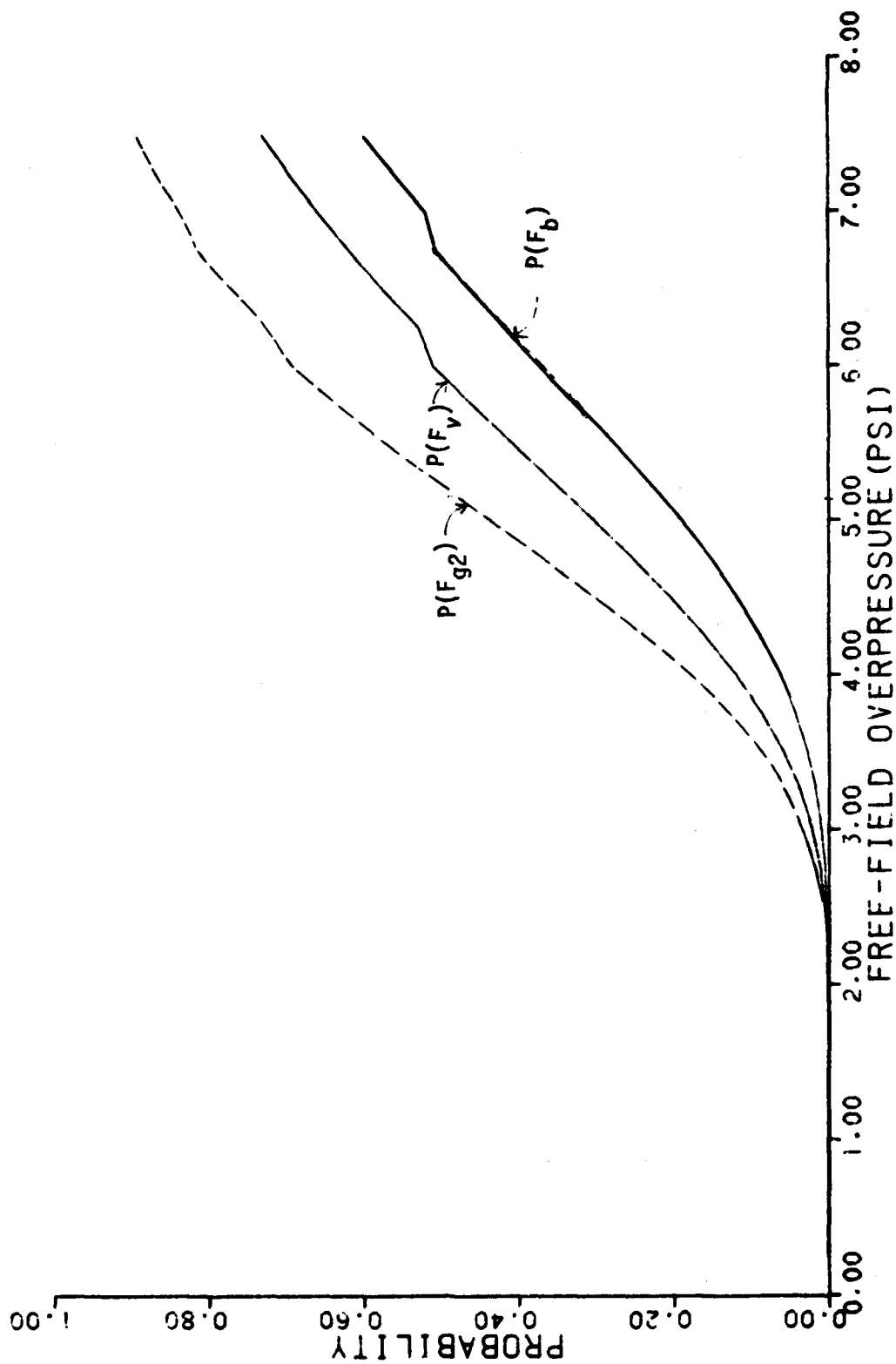


Figure A-10. Probability of girder failure (Part 2).

where P = column load

A = cross-sectional area of column

E = modulus of elasticity

l = unsupported column length

d = least dimension of the rectangular cross-section of the column.

Columns used in this basement consist of 4 - 2 in. x 4 in. boards nailed together. The columns had the following properties:

Cross-section = 3.5 in. x 6.0 in.

Length = 76.0 in.

E = modulus of elasticity = $1.35 (10)^6$ psi.

The coefficient of variation of F_c is obtained as

$$V(F_c) = \left(\frac{\partial F_c}{\partial E} \right)^2 V(E) + \left(\frac{\partial F_c}{\partial d} \right)^2 V(d) \quad (A-19)$$

where $V()$ = variance of the given parameter. In this analysis E and d are considered to be random variables. The length, l, is assumed to be a constant.

$$\Omega_{F_c} = \frac{\sqrt{V(F_c)}}{F_c} = \sqrt{\Omega_E^2 + 4\Omega_d^2} \quad (A-20)$$

$$\Omega_E = 0.20$$

$$\Omega_d = 0.07 \text{ [see } \Omega_h \text{ in equation (A-6)]}$$

$$\Omega_{F_c} = [(0.20)^2 + 4(0.07)^2]^{1/2} = 0.24$$

$$\theta_c = N_{gc} \frac{F_c}{f_c} \quad (A-21)$$

where N_{gc} = correction factor on the column formula, estimated at 0.95 (see "Uncertainty in the Flexure Formula" and Figure A-5)

f_c = column stress due to applied load.

$$\bar{\theta}_c = (0.3) \bar{N}_{gc} E A / [(l/d)^2 \bar{P}_i] \quad (A-22)$$

where \bar{P}_i = applied column load, $i = 1, 5$

$$l/d = 76/3.5 = 21.71$$

$$\bar{\theta}_c = (0.30)(0.95)(1,350,000)(3.5)(6.0) / [21.71)^2 \bar{P}_i] = 17,143 / \bar{P}_i.$$

$$\Omega_{\theta_c} = (\Omega_{gc}^2 + \Omega_{Fc}^2 + \Omega_A^2 + \Omega_{P_i}^2)^{1/2} \quad (A-23)$$

$$\Omega_{gc} = 0.03, \text{ (see A-4)}$$

$$\Omega_{Fc} = 0.24$$

$$\Omega_A = 0.14$$

$$\Omega_{P_i} = 0.20$$

$$\Omega_{\theta_c} = [(0.03)^2 + (0.24)^2 + 0.14^2 + (0.20)^2]^{1/2} = 0.34$$

Expressions for computing failure probabilities for the five columns are given next.

Column	\bar{P}_i	$\bar{\theta}_c$	$P(F_{ci})$	
1	1914.4 \bar{p}	8.9544/ \bar{p}	$1 - \Phi \left(\frac{2.1921 - \ln \bar{p}}{0.34} \right)$	(A-23)
2	5868.6 \bar{p}	2.9211/ \bar{p}	$1 - \Phi \left(\frac{1.0720 - \ln \bar{p}}{0.34} \right)$	(A-24)
3	3251.2 \bar{p}	5.2727/ \bar{p}	$1 - \Phi \left(\frac{1.6625 - \ln \bar{p}}{0.34} \right)$	(A-25)
4	4671.1 \bar{p}	3.6699/ \bar{p}	$1 - \Phi \left(\frac{1.3002 - \ln \bar{p}}{0.34} \right)$	(A-26)
5	4594.4 \bar{p}	3.7312/ \bar{p}	$1 - \Phi \left(\frac{1.3167 - \ln \bar{p}}{0.34} \right)$	(A-27)

Corresponding failure probabilities are shown in Figure A-11. $P(F_c)$ is the column system failure probability assuming that the columns act independent of each other. It was computed using the following expression, see equation (49):

$$P(F_c) = 1 - \prod_{i=1}^S [1 - P(F_{ci})] \quad (A-28)$$

The lower bound is represented by $P(F_{c1})$.

A.1.7.2 Studwall Columns

The studwall columns used in the expedient upgrading of the basement are shown in Figure A-3. See Figure A-1 for the center to center dimensions.

Studwall column cross-section = 1.5 in. x 3.5 in.

Height, $\ell = 76$ in.

$\ell/d = 76/(2 \times 1.5) = 25.33$

Column load: East side, $P_e = 1683\bar{p}$ (see Figure A-4)

West side, $P_w = 1490\bar{p}$

Studwall columns at each side (east or west) are identical. Failure of the column system at each side, therefore, is represented by the failure of one column.

$$\bar{\theta}_c = N_{gc} \bar{F}_c \bar{A}/\bar{P}_i$$

where \bar{F}_c is defined by equation (A-18), thus

$$\bar{\theta}_c = 0.3N_{gc} E A/[(\ell/d)^2 \bar{P}_i], \quad \Omega_{\theta_c} = 0.34$$

For the east side:

$$\bar{\theta}_c = 0.3(0.95)(1,350,000)(1.5)(3.5)/[(25.33)^2 1683\bar{p}] = 1.8706/\bar{p}$$

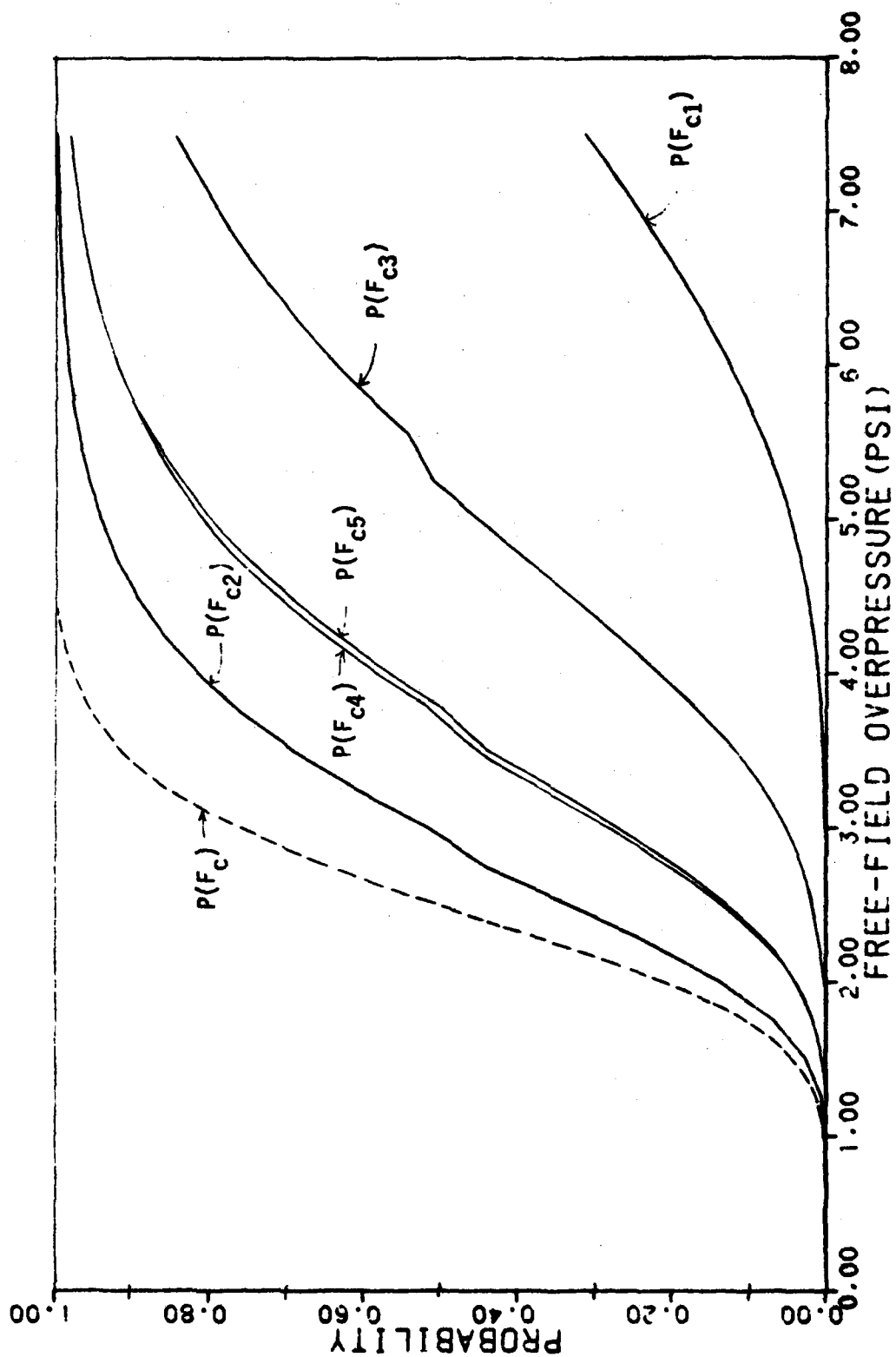


Figure A-11. Probability of column failure.

$$P(F_{se}) = 1 - \Phi \left(\frac{0.6263 - \ln \bar{p}}{0.34} \right) \quad (A-24)$$

For the west side:

$$\theta_c = 0.3(0.95)(1,350,000)(1.5)(3.5)/[(25.33)^2 1490\bar{p}] = 2.1129/\bar{p}$$

$$P(F_{sw}) = 1 - \Phi \left(\frac{0.7481 - \ln \bar{p}}{0.34} \right) \quad (A-25)$$

Results are shown in Figure A-12. For the column system comprising the two studwalls, the lower bound failure probability is given by $P(F_{sw})$. The upper bound is given by $P(F_s)$, which was computed using the expression

$$P(F_s) = 1 - [1 - P(F_{sw})][1 - P(F_{se})] \quad (A-26)$$

A.1.8 Failure Probability of the System

Upper bound values are obtained based on the assumption that the conditions between different components are statistically independent. Lower bound values are based on the perfect correlation assumption between the components.

$P(F^*)$ = upper bound failure probability

$$P(F^*) = 1 - [1 - P(F_j)][1 - P(F_{g1})][1 - P(F_{g2})][1 - P(F_c)][1 - P(F_{sw})][1 - P(F_{se})] \quad (A-27)$$

$P(F')$ = lower bound failure probability

$$P(F') = \max[P(F_j), P(F_{g1}), P(F_{g2}), P(F_c), P(F_{sw}), P(F_{se})] \quad (A-28)$$

Results are plotted in Figure A-13.

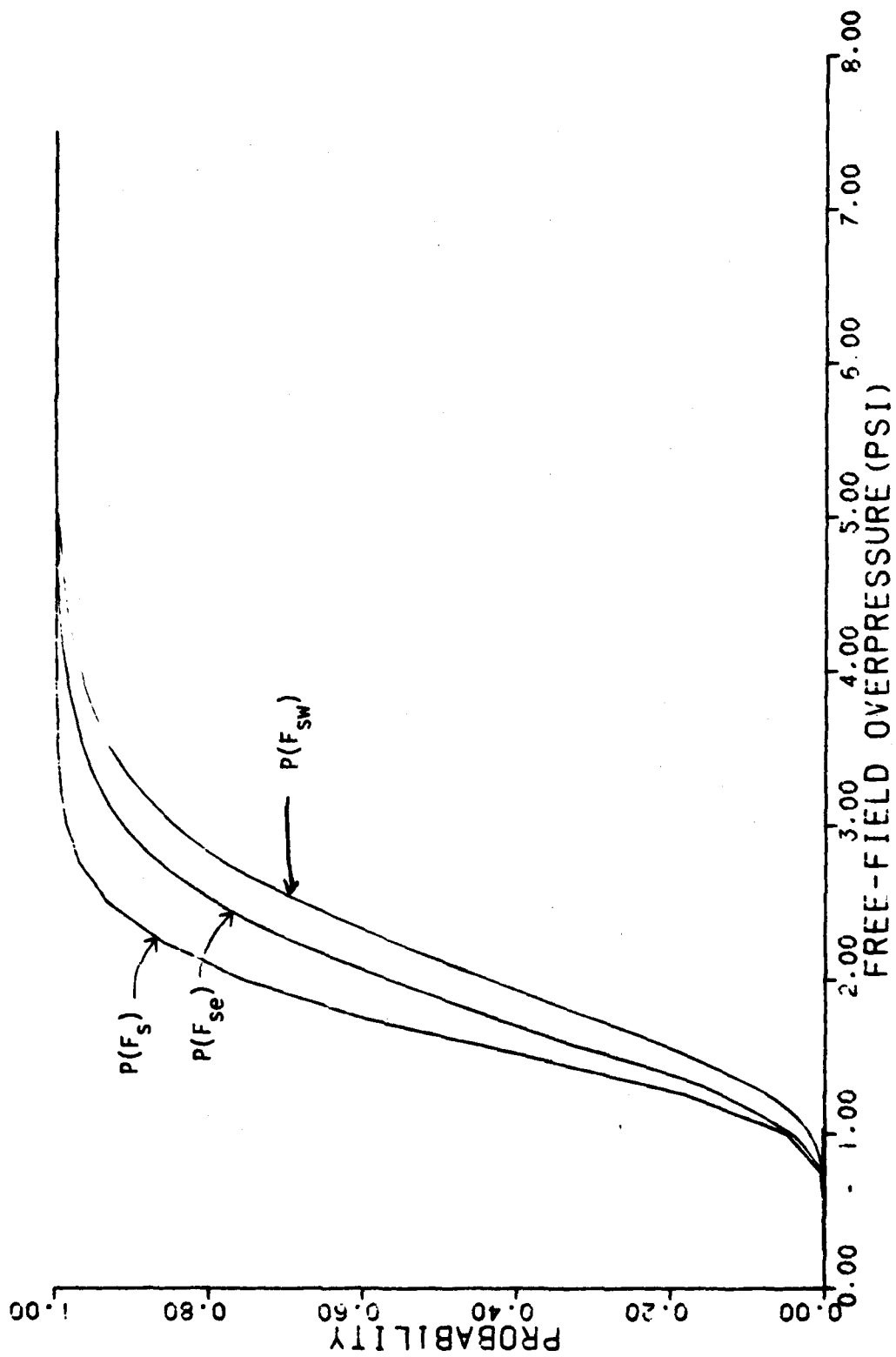


Figure A-12. Failure probability of studwall column system.

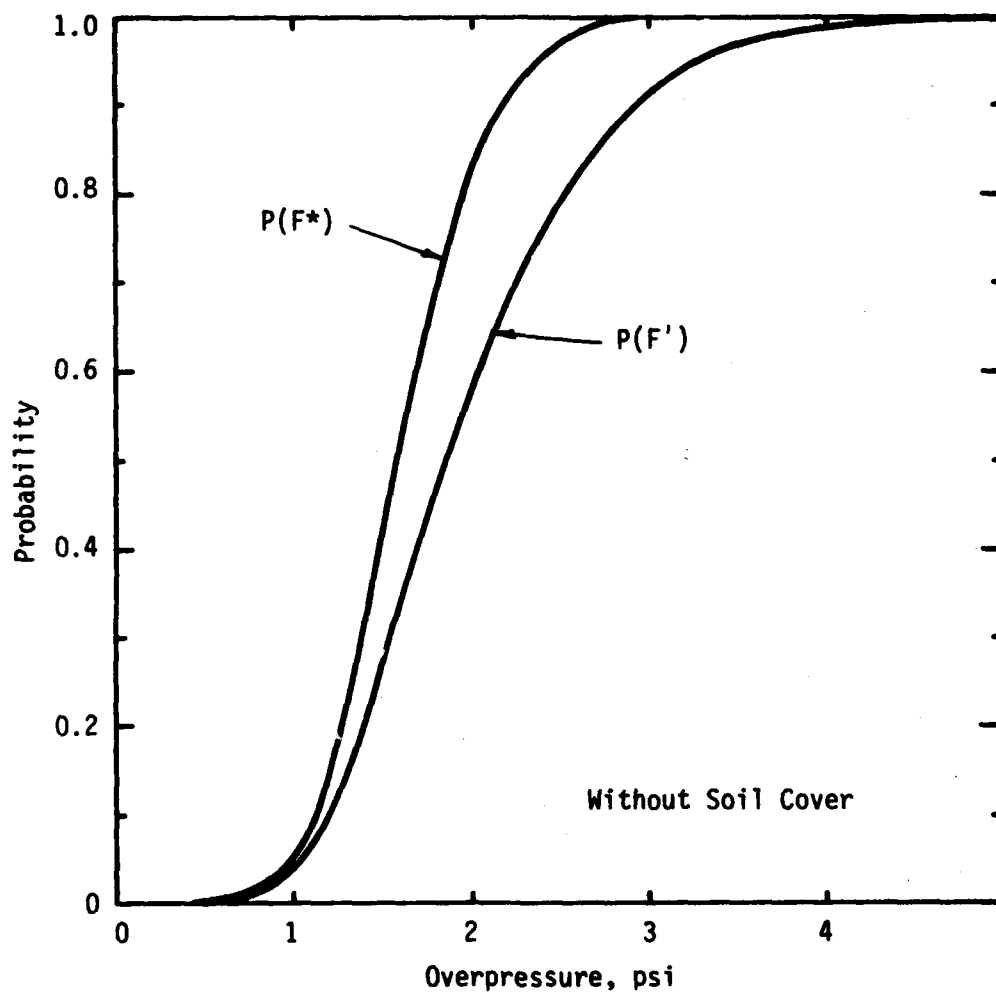


Figure A-13. Probability of floor system failure, upper and lower bound.

A.2 PROBABILITY OF PEOPLE SURVIVAL

As shown in Figure A-13, the relevant range of overpressures is between 0 and about 5 psi. In this range, the dominant casualty mechanism for people in basements is debris from the collapse of the overhead floor system and the upper story. The upper story is expected to fail in the range of 1.5 psi to 2.9 psi. The following failure probabilities are estimated (Ref 24).

<u>Probability of Failure (%)</u>	<u>Overpressure (psi)</u>
10	1.5
50	2.2
90	2.9

Probability of people survival against structural collapse is determined using the theorem of total probabilities (Ref 25) as

$$P(S_{sc}) = P(S|\bar{F})P(\bar{F}) + P(S|F)P(F) \quad (A-29)$$

where $P(S_{sc})$ = probability of people survival against structural collapse

$P(S|\bar{F})$ = probability of people survival given that the structure (floor system) does not fail

$P(\bar{F})$ = probability of structure survival

$P(S|F)$ = probability of people survival given that the structure collapses (fails)

$P(F)$ = probability of structural failure = $1 - P(\bar{F})$.

For this structure, $P(F)$ is given by equations (A-27) and (A-28).

No fatality level casualties are expected prior to the collapse of the floor system and, therefore, $P(S|\bar{F})$ is set equal to 1. Probability of survival given that the structure collapses, $P(S|F)$, is estimated to be 0.5. It is based on the following reasoning.

When the floor system over the basement collapses, the debris is not expected to affect the entire shelter area. Several portions of the basement are expected to be free of debris. People located in these areas will be survivors. At least one half of the total basement area is expected to be

free of debris effects. For people uniformly distributed, the probability of survival is, therefore, estimated as 0.5.

Probability of people survival results are given in Figure A-14. Two cases are considered, i.e., with and without soil cover for radiation protection.

The analysis and results given here represent an update and revision of results given in Reference 20.

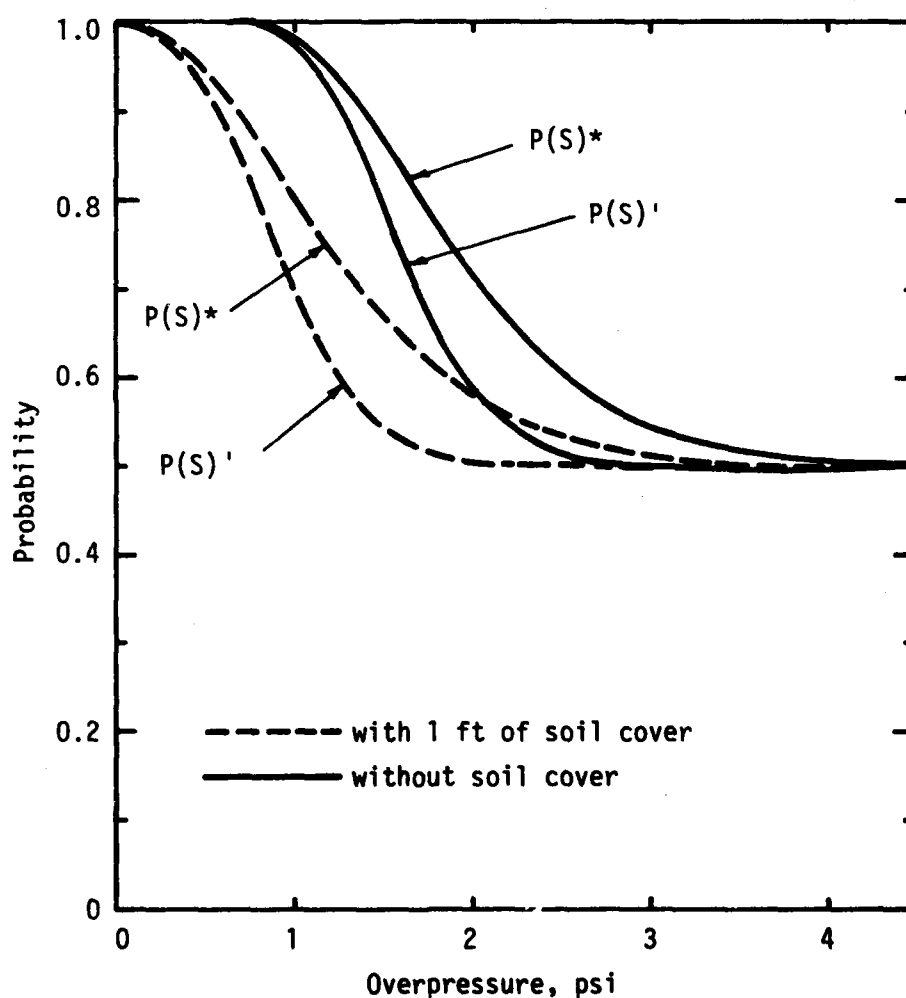


Figure A-14. Probability of people survival, upper and lower bound.

APPENDIX B

PROBABILITY OF PEOPLE SURVIVAL IN UPGRADED BASEMENTS OF SINGLE FAMILY RESIDENCES

B.1 INTRODUCTION

This appendix contains results on the probability of survival of people located in basements of framed, single-family residences when subjected to the blast effects of a single, megaton yield unclear weapon detonated near the ground surface. Four buildings are considered. In each, the basement is expediently upgraded against blast effects by providing additional supports for the joist floor system. Additional supports are either studwalls or post and beam (girder and column) systems. Six cases are considered:

<u>Building Name</u>	<u>Type of Upgrading</u>
1. Dunes House	Studwall (see Figure A-3) Girder and Column (see Figure B-1)
2. West House	Studwall
3. Park House	Studwall
4. Tea Pot House	Studwall Girder and Column

The analysis considering the Dunes House with the studwall upgrading is described in Appendix A, which also contains the probability of structural failure and people survival results. The remaining cases outlined above are summarized in the following sections.

B.2 GENERAL ASSUMPTIONS

- (1) The basement framing system (joists, girders, columns) for each building is assumed to consist of "Jack Pine" whose properties are given in Table A-1.
- (2) The expedient upgrading system (studwall, girder and column) is also assumed to consist of Jack Pine.
- (3) The upper story in each case is assumed to fail and be removed by the blast in the overpressure range of 1.5 to 2.9 psi (Ref 24).
- (4) There is no interaction between the upper story and the basement framing systems, i.e., the upper story is assumed to cause no damage to the basement while being broken and removed by the blast loading.

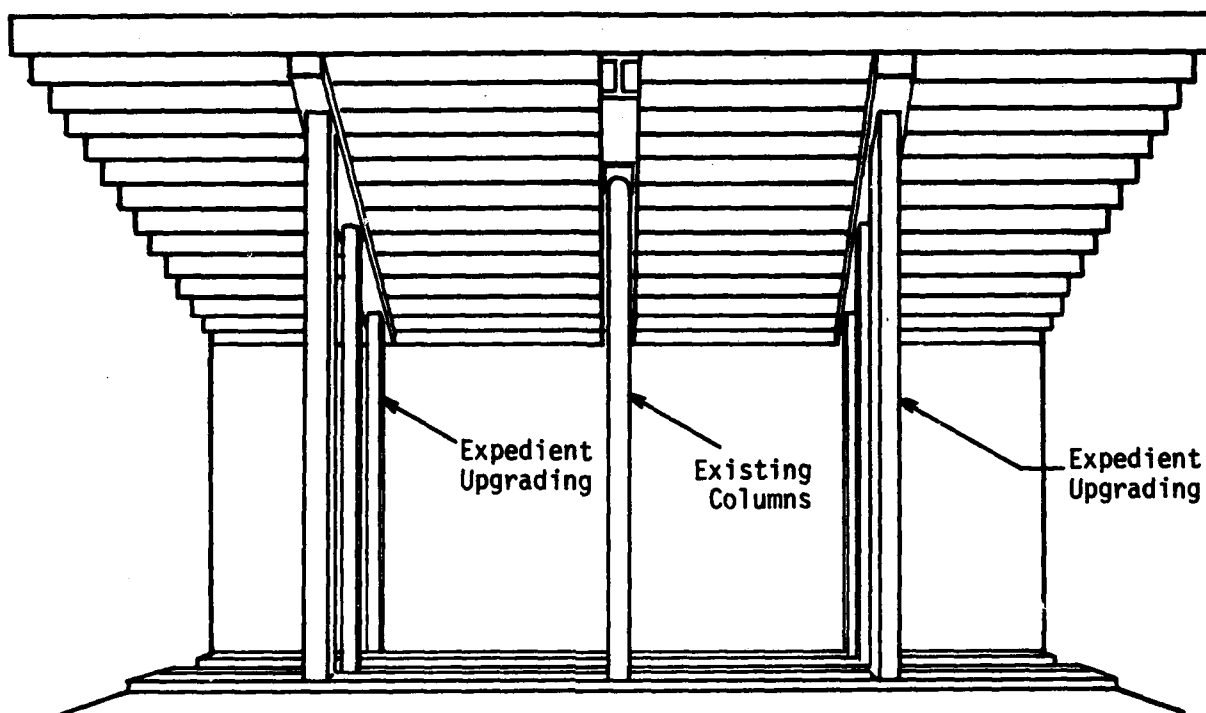


Figure B-1. Post and beam expedient upgrading concept.

- (5) Dead load of the floor system over the basement is neglected. This amounts to approximately 15 psf.
- (6) People are assumed to be uniformly distributed in all basement areas.
- (7) The only casualty mechanism considered in the analysis is debris from the breakup and collapse of the floor system into the basement area.
- (8) Basement walls are assumed to be stronger than all other structural components and are, therefore, assumed not to fail. Analyses to determine failure overpressures for the peripheral basement walls were not performed. However, based on the results of full-scale field tests (Ref 26) this is a reasonable assumption in this case.

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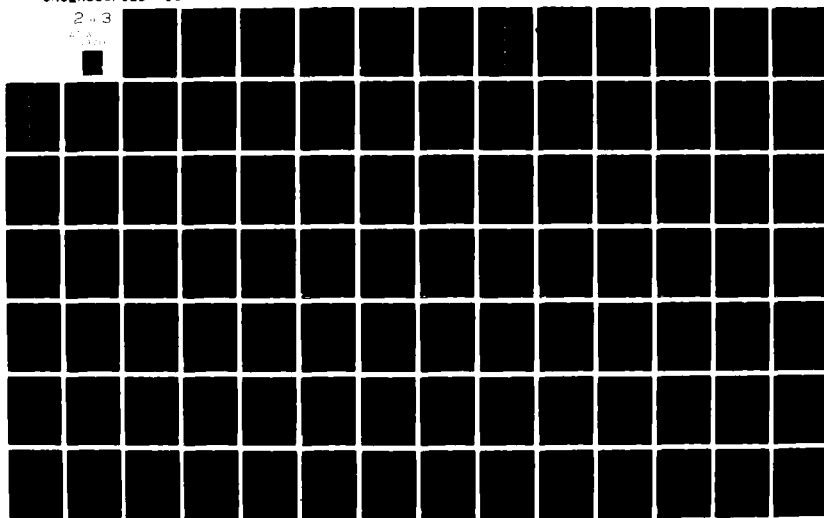
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B.3 DUNES HOUSE

The basement of this house is described in Appendix A where it was analyzed with an expedient blast upgrading consisting of a studwall in each of the two joist spans (see Figure A-2). In this section the expedient upgrading consists of the "girder and column" concept (see Figure B-1) with and without soil cover for nuclear radiation protection. The size (cross-section and length) of girders used is assumed to be the same as that of the existing girder. Columns are also assumed to be of the same size and number as the existing columns and are assumed to be identically spaced and supported. The analysis was performed along the lines described in Appendix A. Results are summarized.

B.3.1 Failure Probabilities

Failure probabilities for all structural components making up the floor system and its supporting elements, except the basement walls, are given in Figures B-2 through B-4. Figure B-2 illustrates failure probabilities for the joists and the existing girder. Failure probabilities for existing columns and columns used with the expedient upgrading are given in Figure B-3. Failure probabilities for the two sets of girders used in the expedient upgrading are given in Figure B-4. Each of the curves is an upper bound on the particular failure probability and was determined using equation (49). The bounds on the probability of failure of this expediently upgraded floor system are given in Figure B-5 for the case without soil cover. In this figure, $P(F^*)$, the upper bound, is based on equation (49) and $P(F')$, the lower bound, represents the failure probability of upgrading column 2 located in the east joist span (see Figure A-2).

B.3.2 People Survival Probabilities

Probabilities of people survival are given in Figure B-6, which includes two cases, i.e., with and without soil cover. Probability of survival is against the effects of debris produced by the breakup of the floor system.

B.4 WEST HOUSE

This is an existing single-family dwelling whose basement floor plan is shown in Figure B-7. The floor system over the basement consists of a subfloor

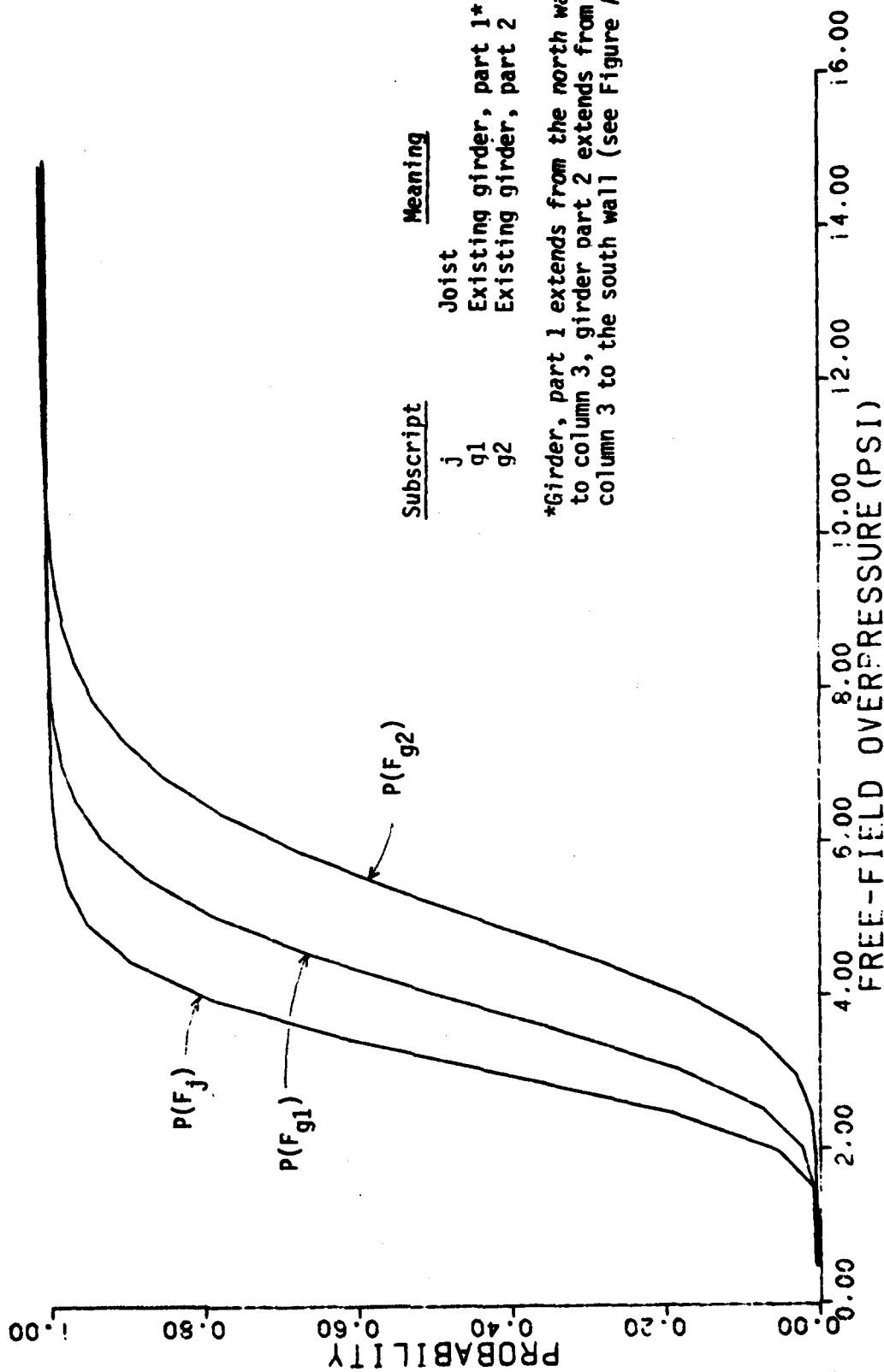


Figure B-2. Joists and existing girder failure Probabilities, Dunes House.

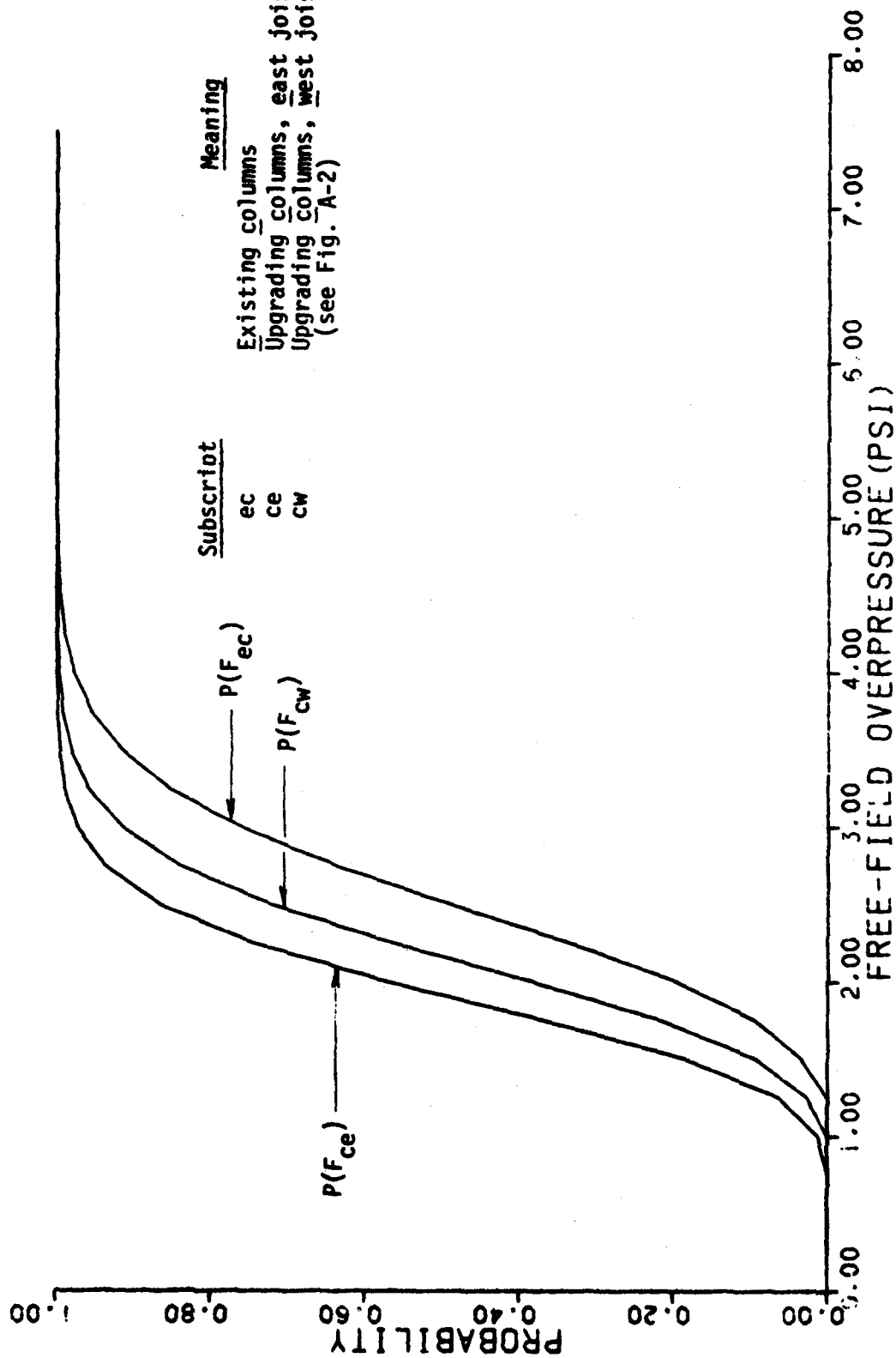


Figure B-3. Column failure probabilities, Dunes House.

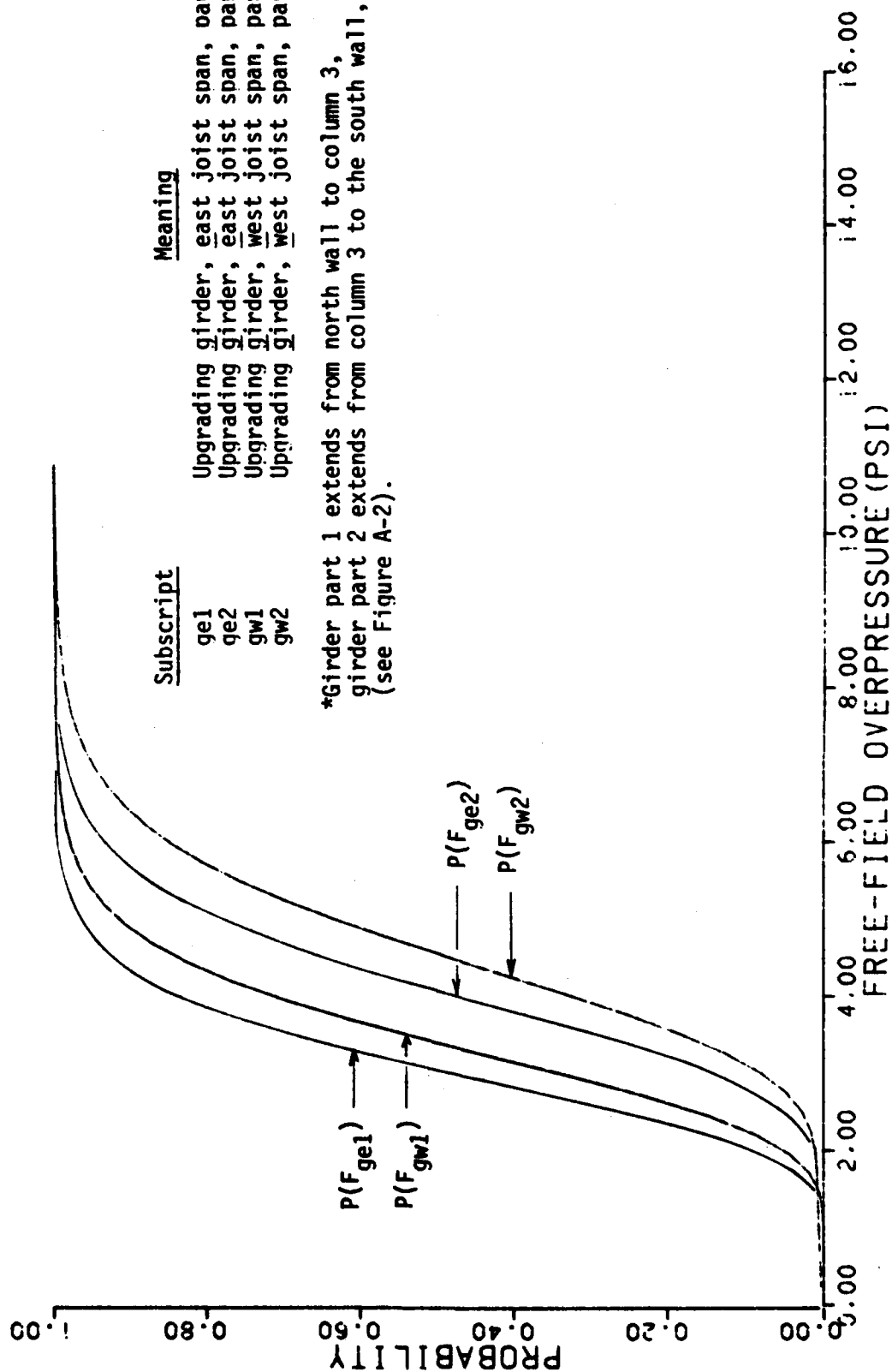


Figure B-4. Upgrading girders failure probabilities, Dunes House.

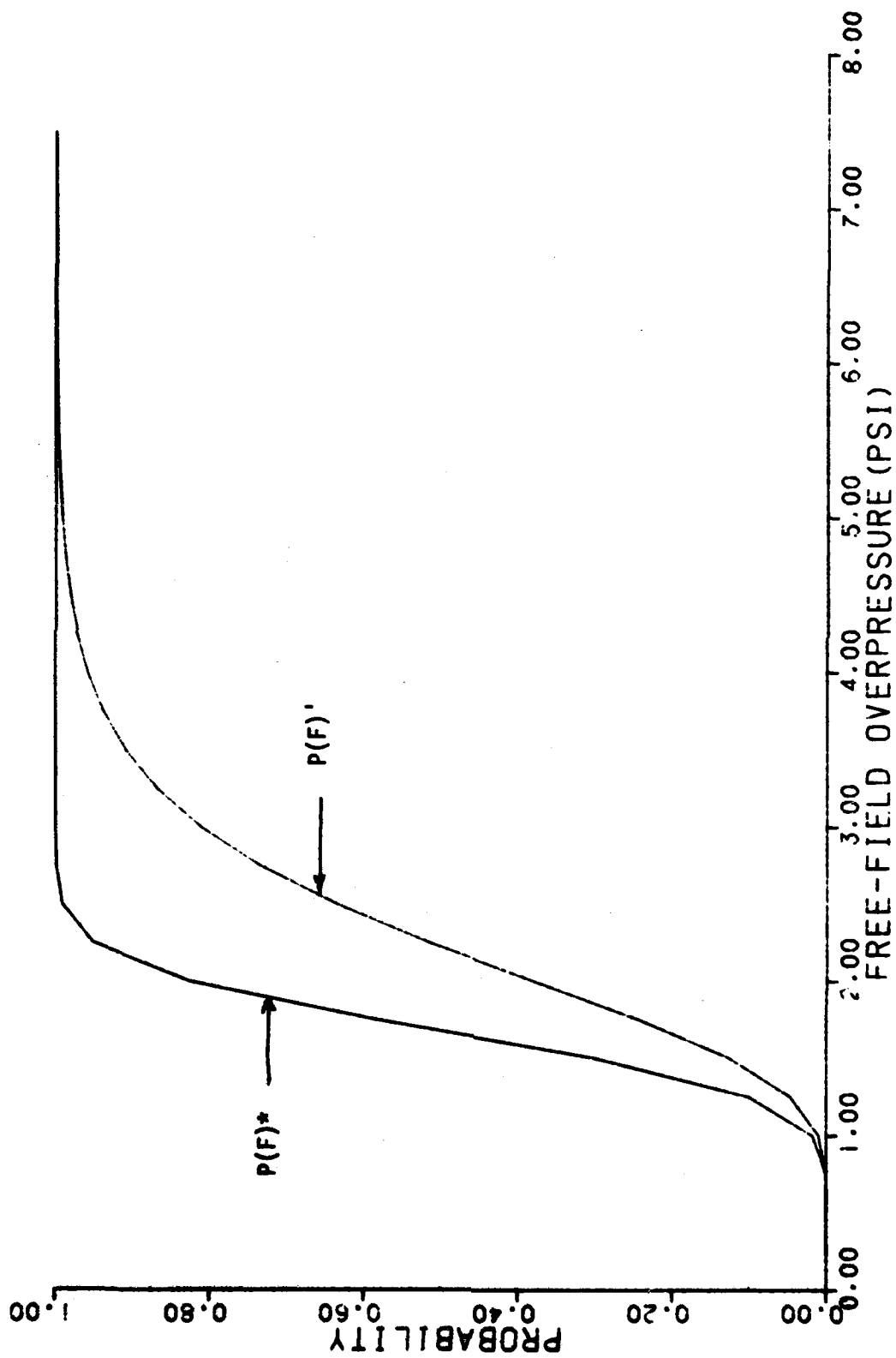


Figure B-5. Probability of floor system failure, upper and lower bound, Dunes House.

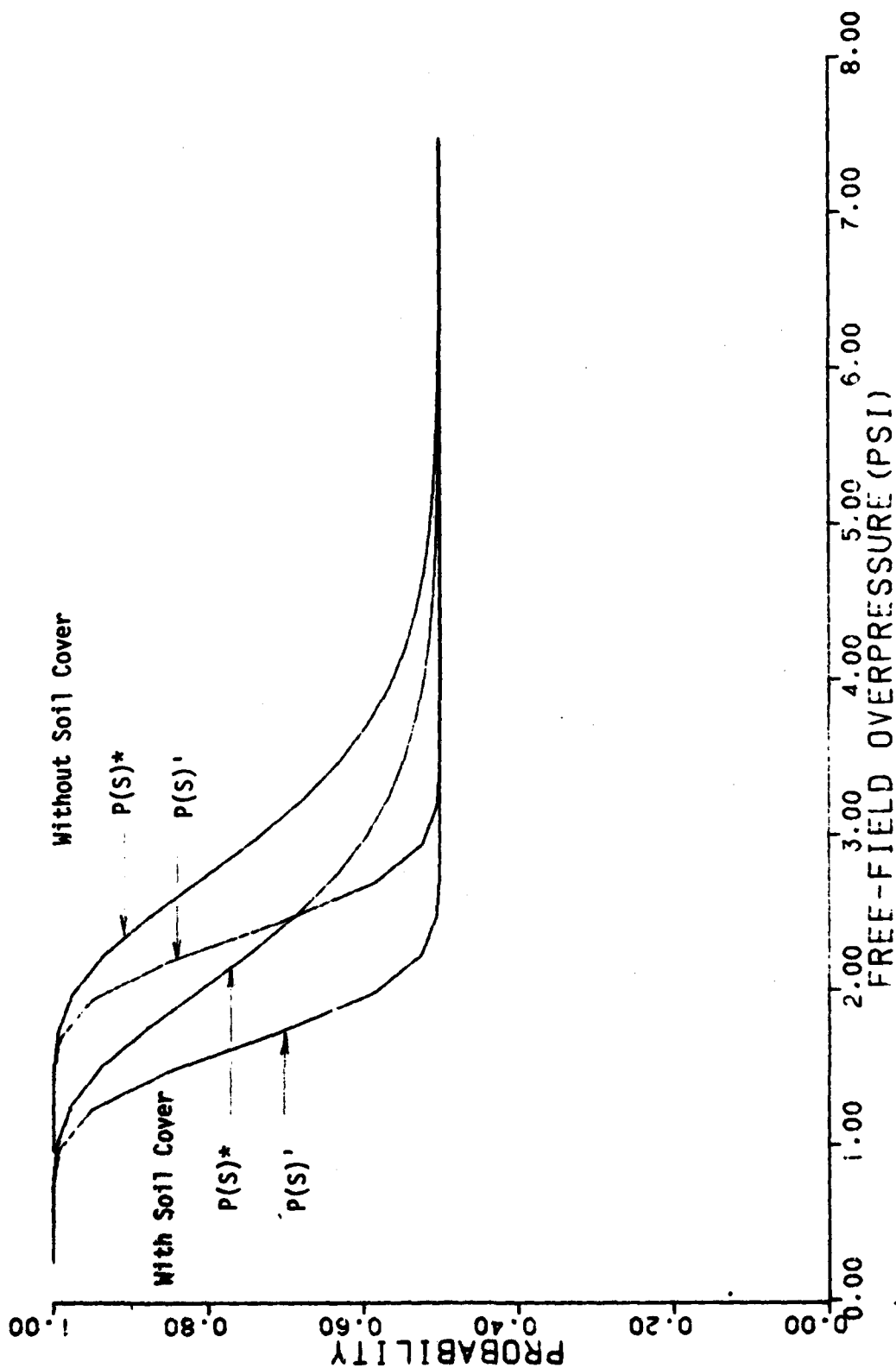


Figure B-6. Probability of people survival, upper and lower bound, Dunes House.

45' 6

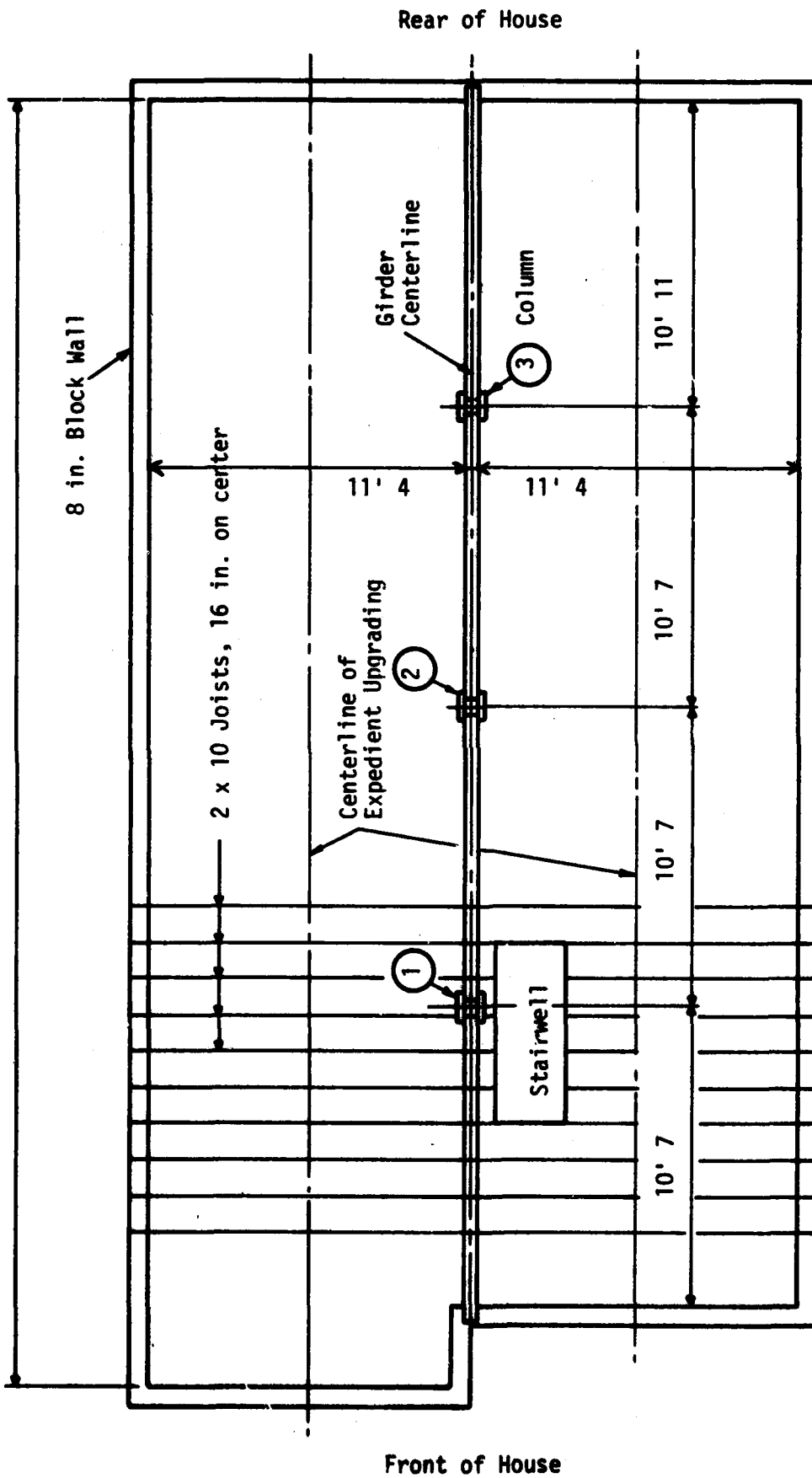


Figure B-7. West House, plan.

and a finish floor supported by 2 in. x 10 in. joists spaced at 16 in. The two joist spans are simply supported. The existing girder is 6 in. x 10 in. and consists of two parts. Part 1 extends from the front wall of the house to column 2. Part 2 extends from column 2 to the rear wall of the house. The three columns have a cross-section of 6 in. by 6 in. Their unsupported length is 77 in.

The floor system is assumed to be upgraded using studwalls in each of the two joist spans. The studs are 2 in. x 4 in. and are spaced at 16 in. as are the joists. Their total height is 70 in. They are braced at half-height as shown in Figure A-3. The analysis was performed along the lines described in Appendix A. Results are summarized.

B.4.1 Failure Probabilities

Failure probabilities for the joists and the two girders are given in Figure B-8. Failure probabilities for the columns and studwalls are given in Figure B-9. Each of these curves is an upper bound on the particular probability and was determined using equation (49).

The bounds on the probability of failure of the whole floor system, including columns and studwalls, are given in Figure B-10. $P(F)^*$, the upper bound probability of failure of the system, was obtained using equation (49) and $P(F)'$, the lower bound, represents the failure probability of the studwalls.

B.4.2 People Survival Probabilities

Probabilities of people survival against the effects of debris from the collapse of the floor system into the basement are given in Figure B-11. Two cases are considered, i.e., with and without soil cover for radiation protection. Probability of survival is against the effects of debris produced by the collapse of the floor system into the basement.

B.5 PARK HOUSE

This is an existing residence whose basement floor plan is shown in Figure B-12. The floor system over the basement consists of a subfloor and a finish floor supported by 2 in. x 8 in. joists spaced at 12 in. on center.

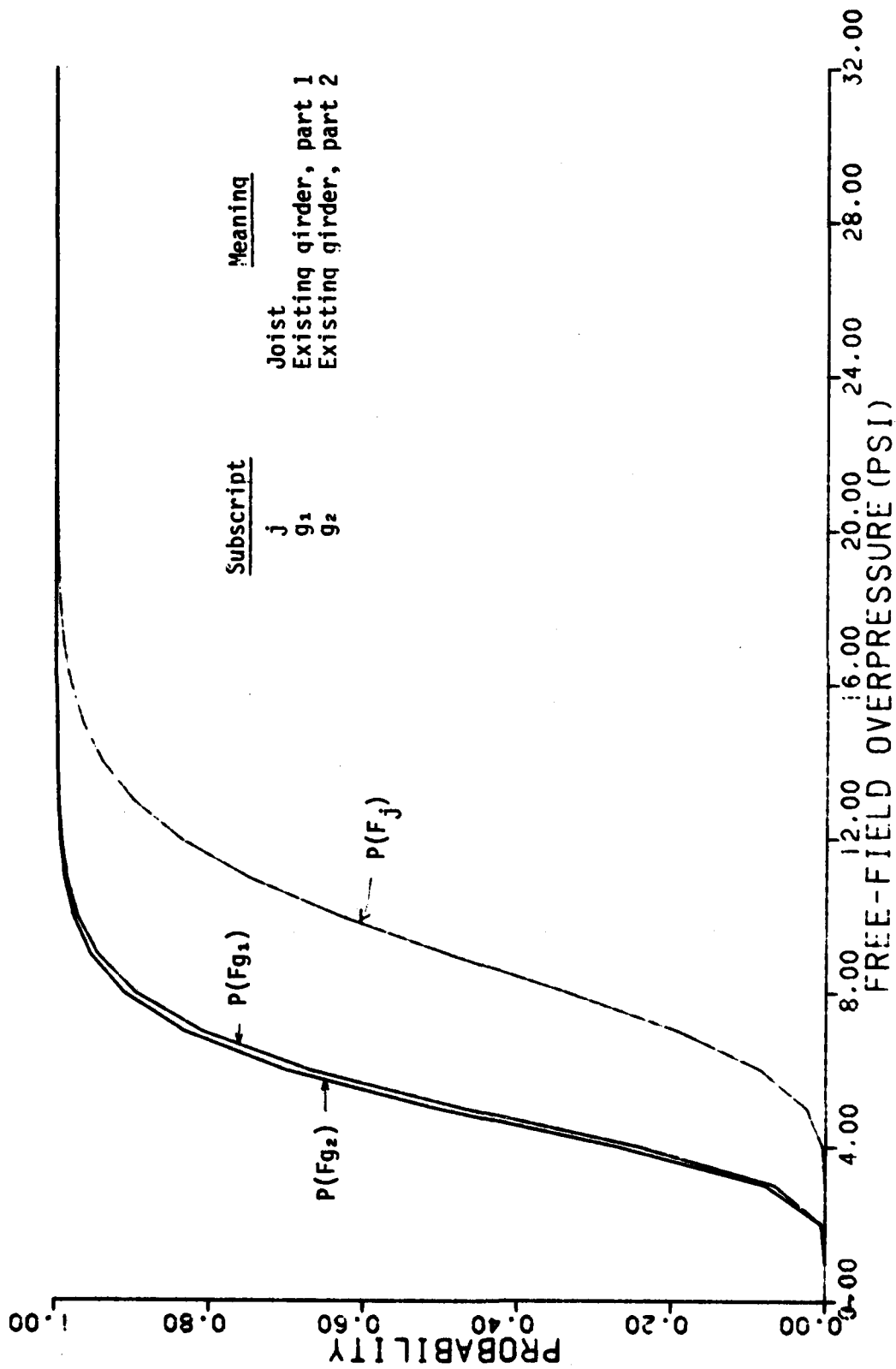


Figure B-8. Failure probabilities for the joists, Girder 1 and Girder 2, West House.

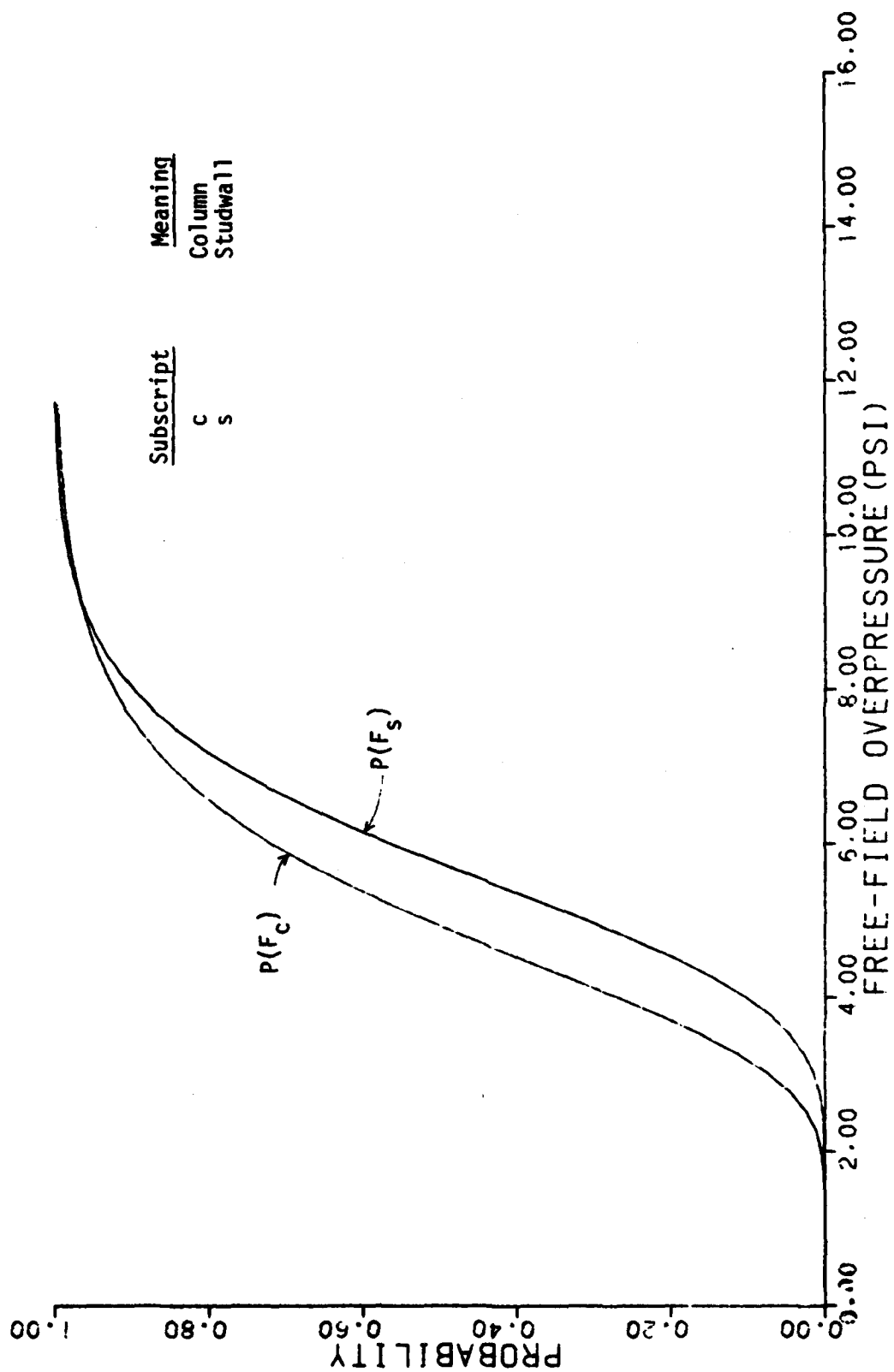


Figure B-9. Failure probabilities for columns and studwalls, West House.

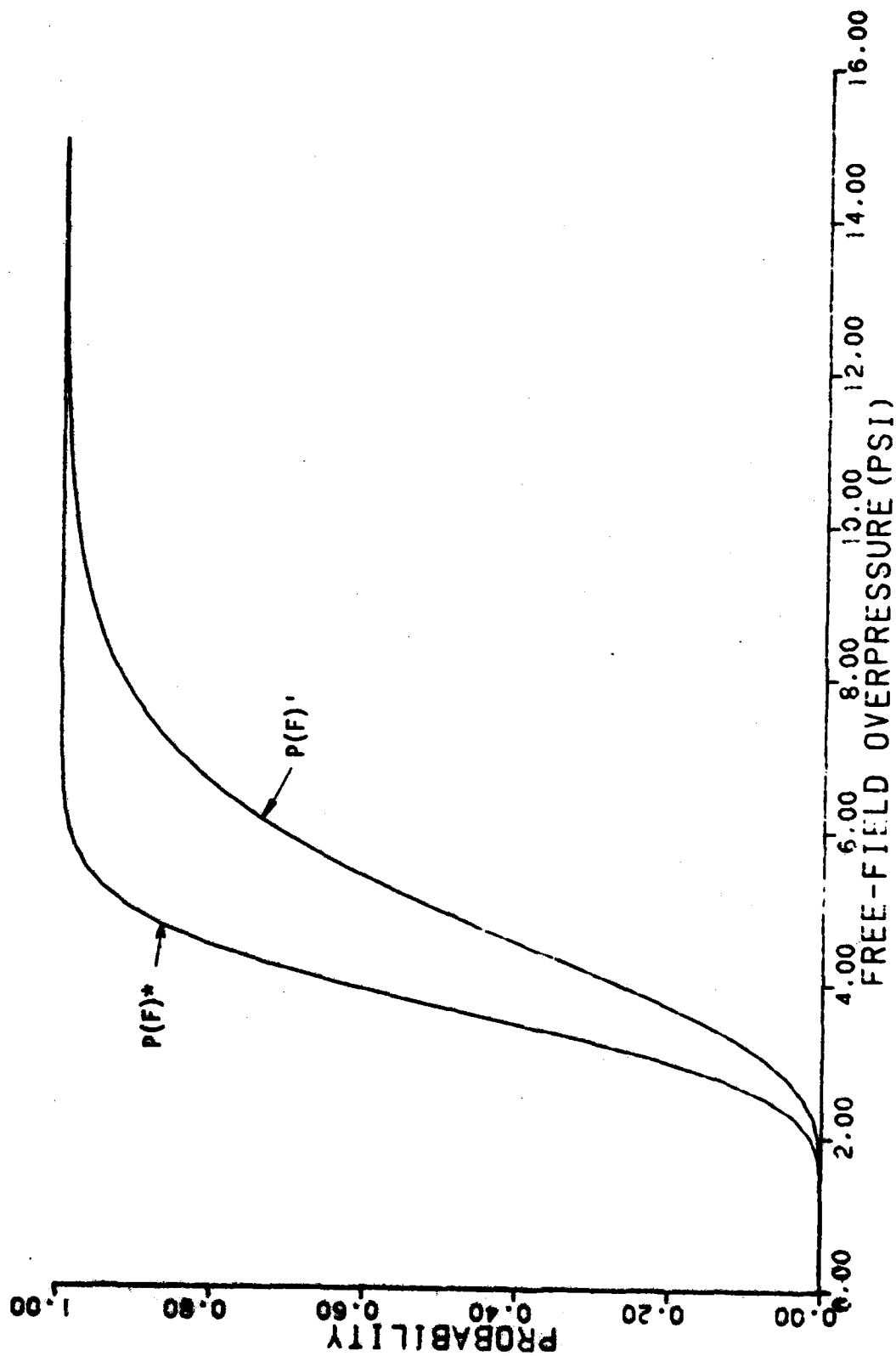


Figure B-10. Probability of floor system failure, upper and lower bound, West House.

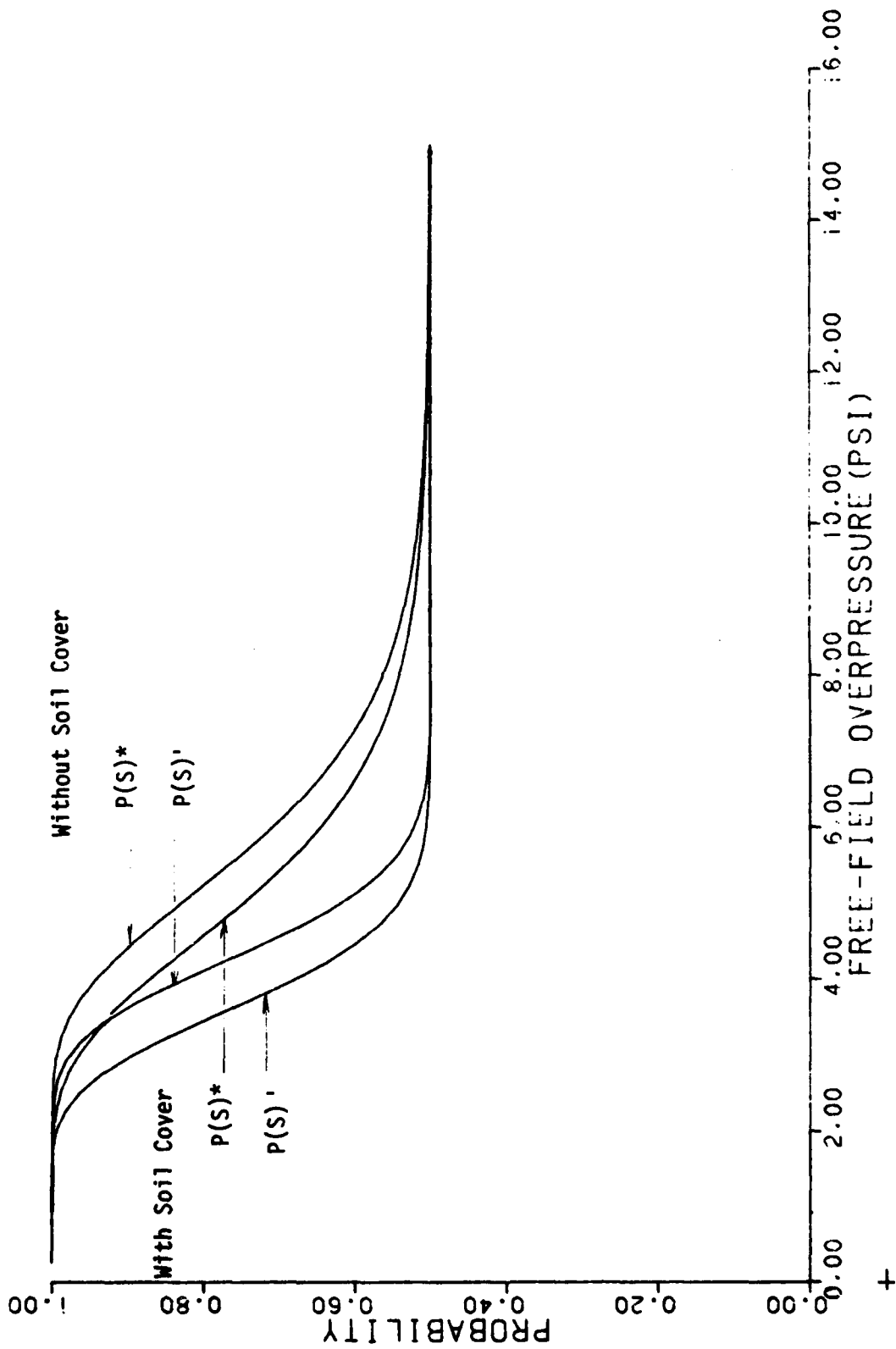


Figure B-11. Probability of people survival, upper and lower bound, West House.

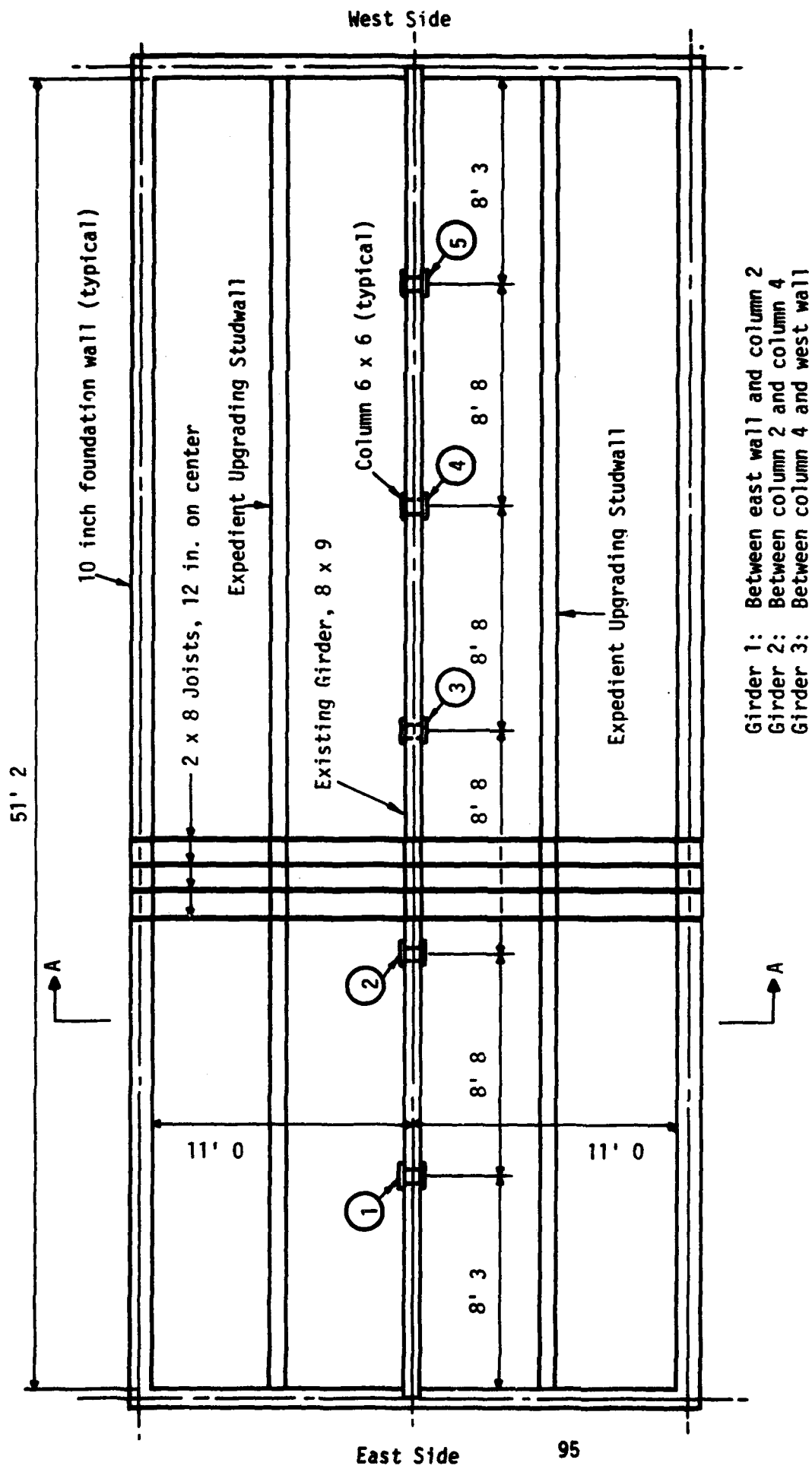


Figure B-12. Park House, plan.

The joists are continuous over the centrally located girder and columns. The girder is 8 in. x 9 in. and consists of three separate parts. The first part spans from the east wall to column 2 (see Fig. B-12), the second part spans from column 2 to column 4, and the third part spans from column 4 to the west wall. The five wood columns have a cross-section of 6 in. x 6 in. Their unsupported length is approximately 7 ft 8½ in.

The floor system is assumed to be expediently upgraded using studwalls in each of the two joist spans. This expedient upgrading is illustrated in Figures B-13 and B-14. The studs are 2 in. x 4 in. and are spaced at 12 in., i.e., one under each joist. Their total height is shown in Figure B-14 and they are braced at half-height. In addition to structurally upgrading the floor, the basement shelter is also assumed to be mounded with soil up to the top of the floor as shown in Figure B-13.

The analysis was performed along the lines described in Appendix A. Results are summarized.

B.5.1 Failure Probabilities

Failure probabilities for the joists and the three timber girders supporting them are shown in Figure B-15. The combined failure probabilities for the columns are given in Figure B-16 together with the failure probability of the studwalls. Each of these curves represents an upper bound and was determined using equation (49).

The bounds on the probability of failure of the whole floor system, including columns and studwalls, are given in Figure B-17. $P(F)^*$, the upper bound probability of failure of the system, was obtained using equation (49). $P(F)'$, the lower bound for the system, is the failure probability of the studwalls.

B.5.2 People Survival Probabilities

Probabilities of people survival against the effects of debris from the collapse of the floor system into the basement are given in Figure B-18. Two cases are considered, i.e., with and without soil cover for radiation protection.

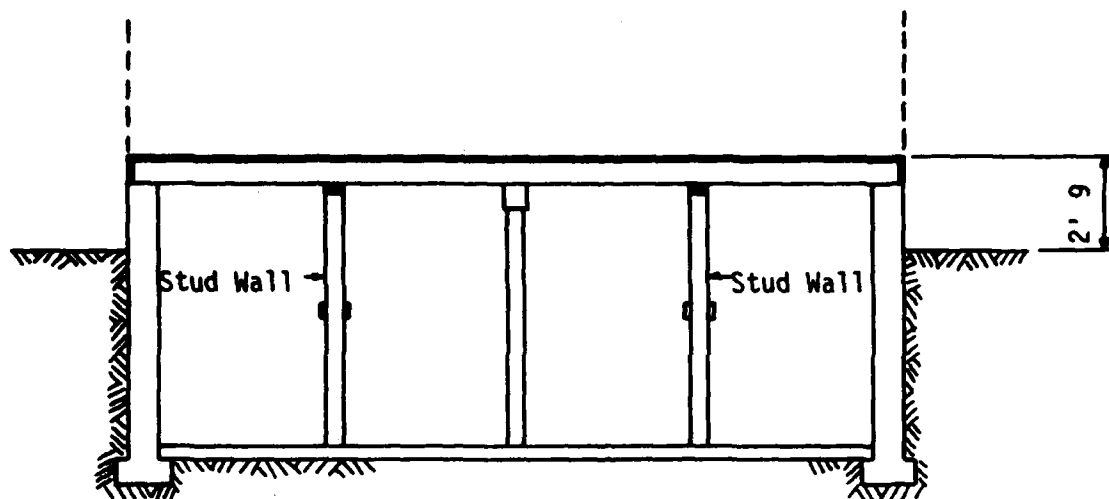


Figure B-13. Elevation (Section A-A).

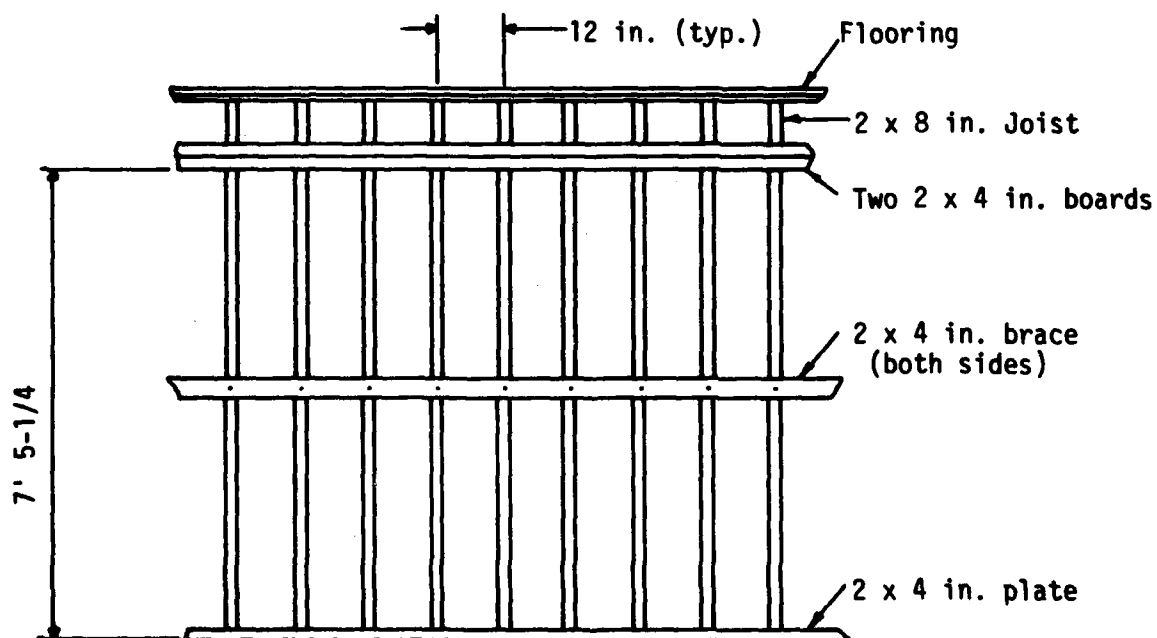


Figure B-14. Studwall expedient upgrading.

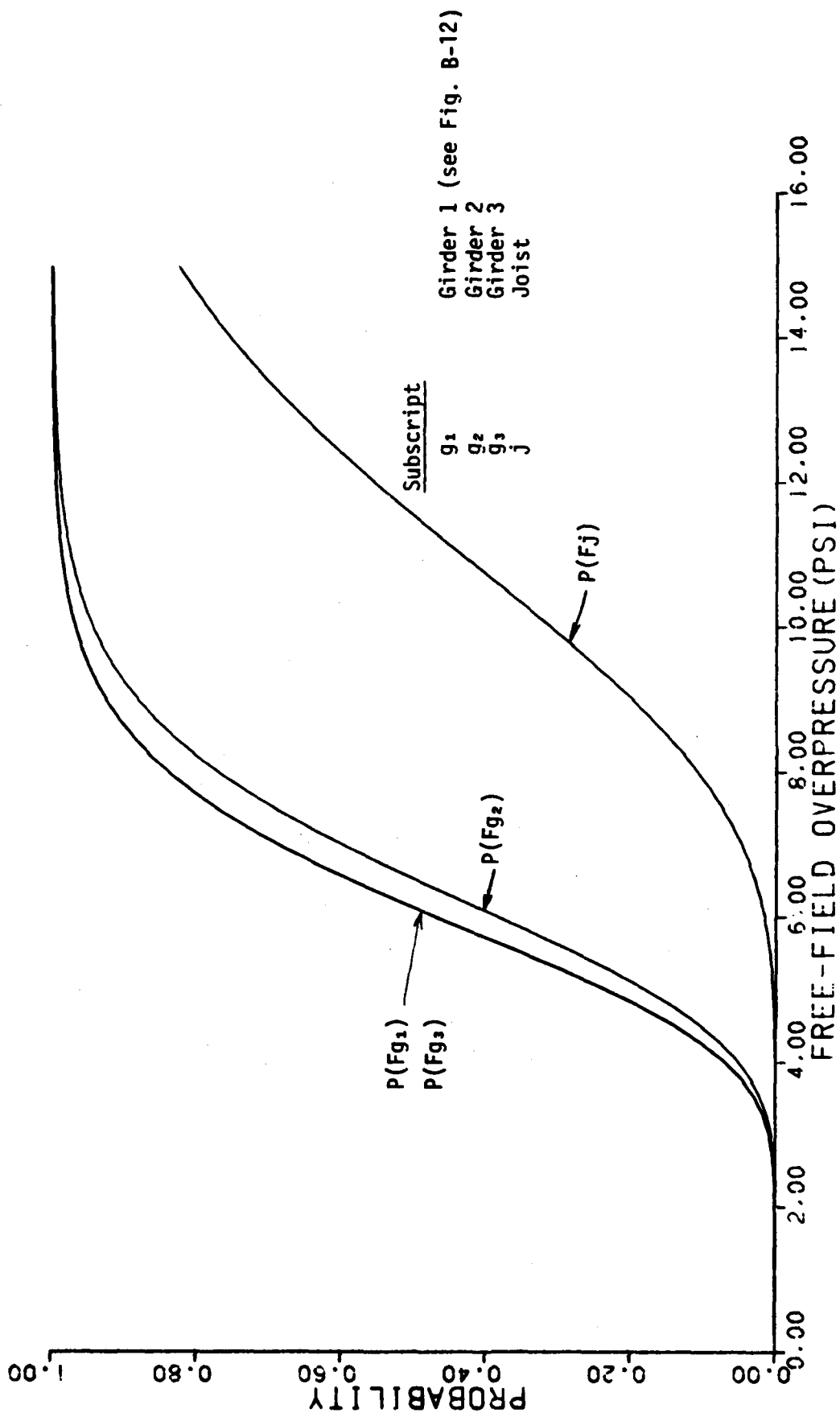


Figure B-15. Failure probabilities of joists and existing girders, Park House.

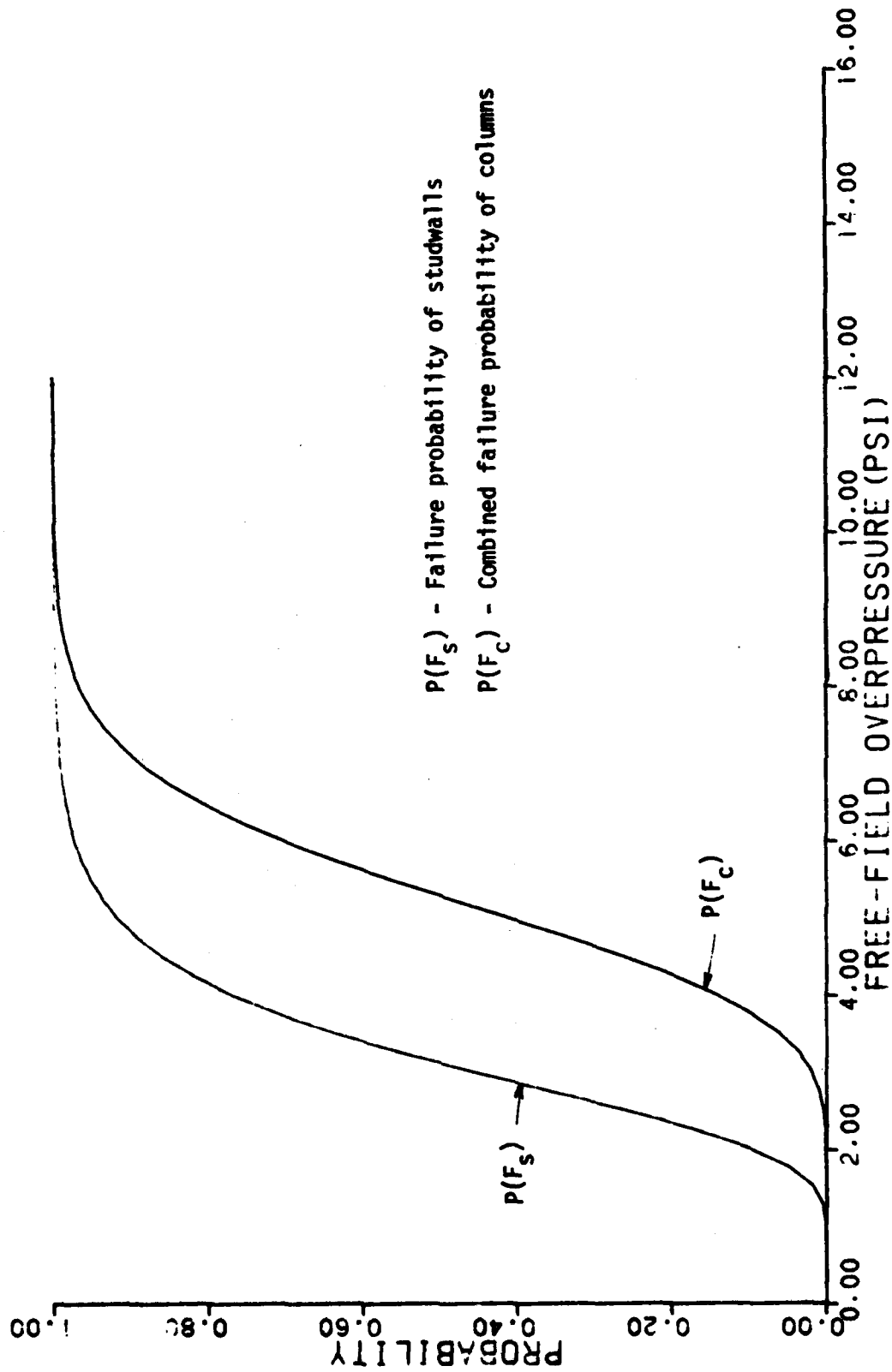


Figure B-16. Failure probabilities of columns and studwalls, Park House.

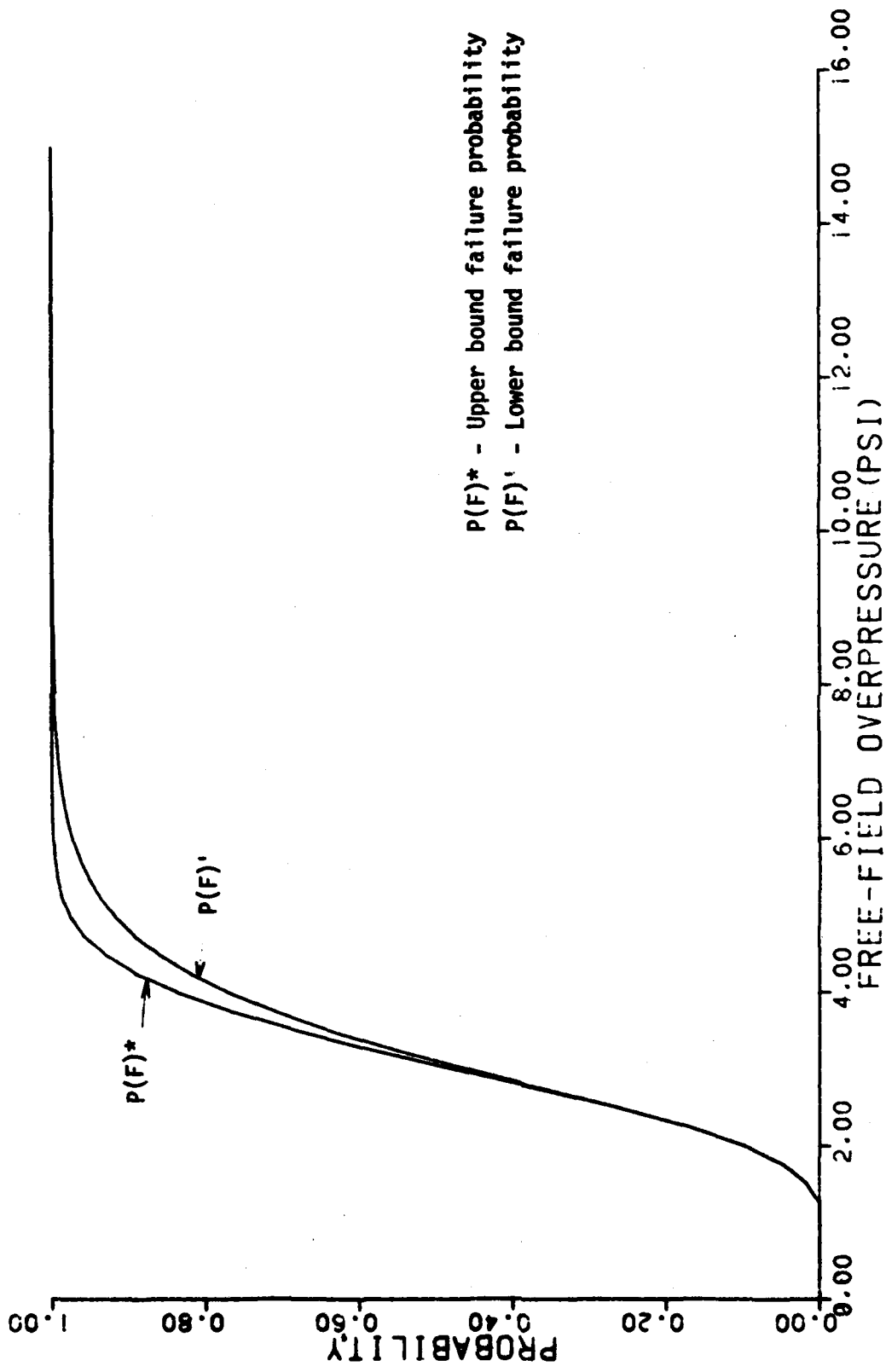


Figure B-17. Probability of floor system failure, upper and lower bound, Park House.

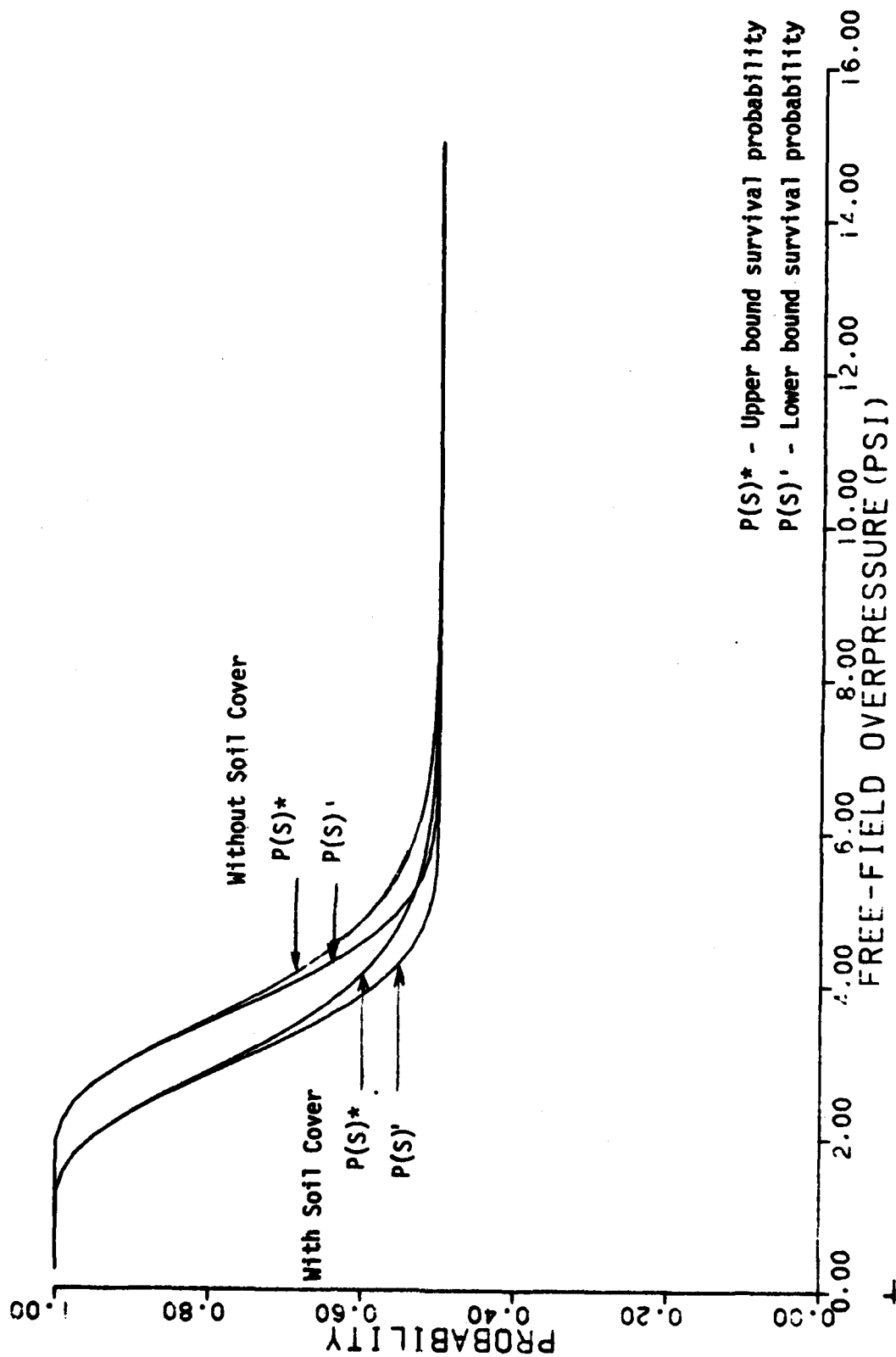


Figure B-18. Probability of people survival, upper and lower bound, Dark House.

B.6 TEA POT HOUSE

This is a two-story, single-family residence originally constructed and tested in Nevada (Ref 26). This building has a full basement with a back entrance, an entrance from the house, and six window wells. The basement plan is shown in Figure B-19.

The floor over the basement consists of a subfloor and a finish floor supported by 2 in. x 8 in. joists spaced at 16 in. centers. The joists (assumed to be continuous over the 33 ft 4 in. length of the house) are supported by two 6 in. x 8 in. girders and the basement walls. The two girders are supported by four steel pipe columns and the basement walls. The peripheral basement walls are made of concrete block. Two expedient upgrading schemes were considered and are described as follows:

a. Scheme 1

The two long (13 ft 4 in.) joist spans were each assumed to be supported by a studwall located halfway between the columns and the walls. This concept calls for a 2 in. x 4 in. stud under each joist.

Entranceways into the basement are assumed to be closed (blocked) by means of expedient blast closures. Window glass is assumed to be removed and the openings are also assumed to be blocked by means of expedient blast closures.

The basement is mounded with soil on the outside up to the first floor level, about 2 ft. One foot of soil is assumed to be placed on the first floor for fallout radiation protection.

b. Scheme 2

This expedient upgrading scheme is the same as the first scheme except that instead of a studwall, the two joist spans are assumed to be upgraded by girders and columns located halfway between the existing columns and the walls. The girder is assumed to be of the same size and the same material as the existing girder. The columns consist of "Southern Pine," have a 6 in. x 4 in. cross-section, are 8 ft long, and have the following properties with respect to an axial load:

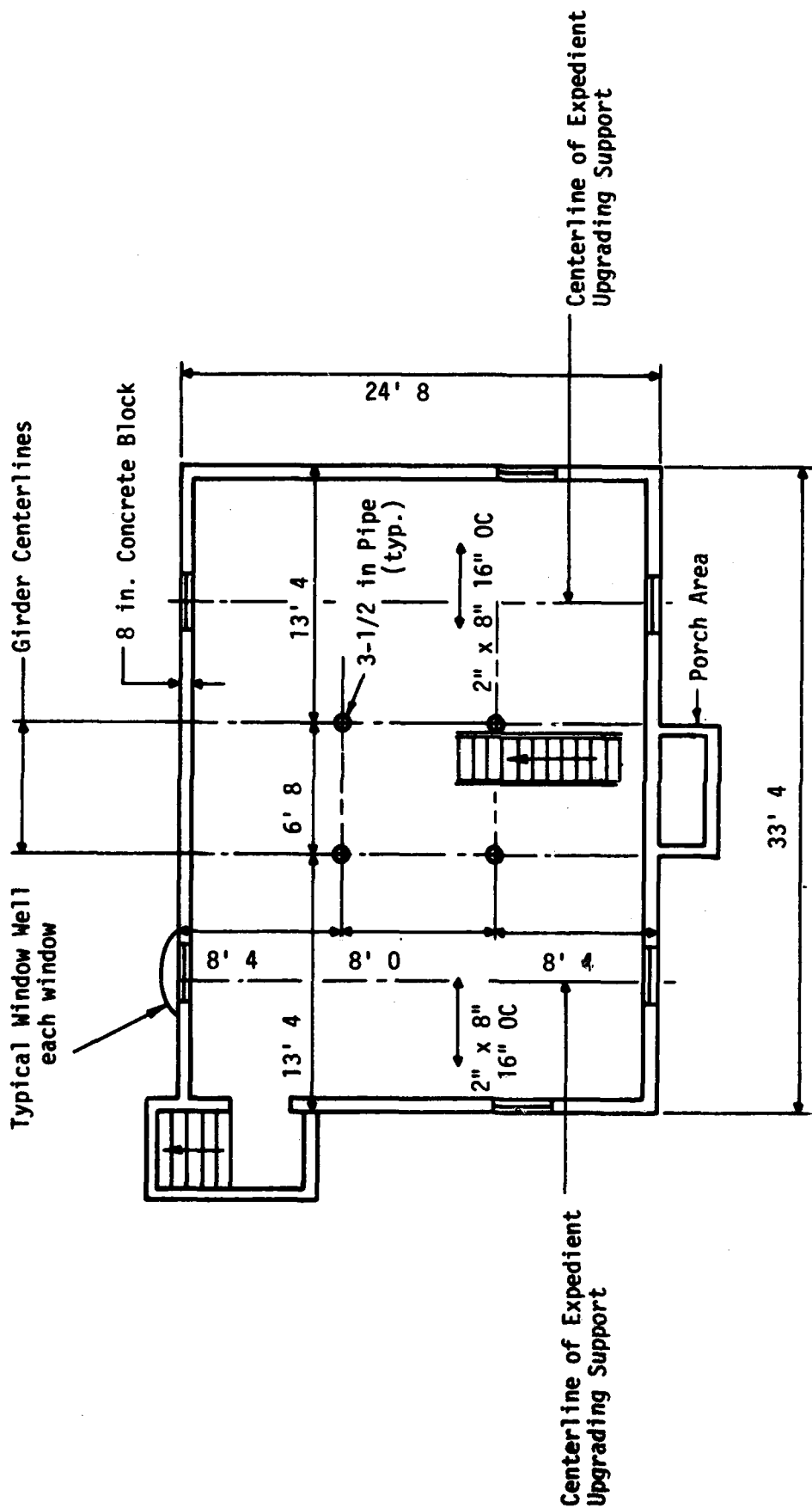


Figure B-19. Tea Pot House basement plan.

F_c (compression parallel to the grain) = 1350 psi

E (modulus of elasticity) = 1,700,000 psi.

Upgrading columns are assumed to have the same spacing as the existing columns. This concept is illustrated in general in Figure B-1.

B.6.1 Failure Probabilities, Scheme 1, Studwall Upgrading

Failure probabilities for the joists and existing girders are shown in Figure B-20. Failure probabilities for existing columns and the studwalls used for upgrading are shown in Figure B-21. Upper and lower bounds on the failure probability of the system as a whole is shown in Figure B-22. In this case the lower bound is the failure probability of the studwalls also shown in Figure B-21. The upper bound was computed using equation (49).

B.6.2 People Survival Probabilities, Scheme 1, Studwall Upgrading

People survival probabilities are presented in Figure B-23 and include two cases, i.e., with and without soil cover for fallout radiation protection. Probability of survival is against the effects of debris produced by the breakup of the floor system over the basement.

B.6.3 Failure Probabilities, Scheme 2, Girder and Column Upgrading

Failure probabilities for the joists, the existing girders, and the girders used in the expedient upgrading are given in Figure B-24. Failure probabilities for existing columns and the columns used in the expedient upgrading are shown in Figure B-25. Upper and lower bounds on the failure probability of the system as a whole are shown in Figure B-26. In this case the lower bound is the failure probability of the columns used in the expedient upgrading. This is also shown in Figure B-25. The upper bound was computed using equation (49).

B.6.4 People Survival Probabilities, Scheme 2, Girder and Column Upgrading

People survival probabilities for this concept are given in Figure B-27. Two cases are considered, i.e., with and without soil cover for fallout radiation protection. Probability of survival is against the effects of debris produced by the breakup of the floor system over the basement. It is evident that the difference between the upper and the lower bounds on the probability of survival is negligible in this case.

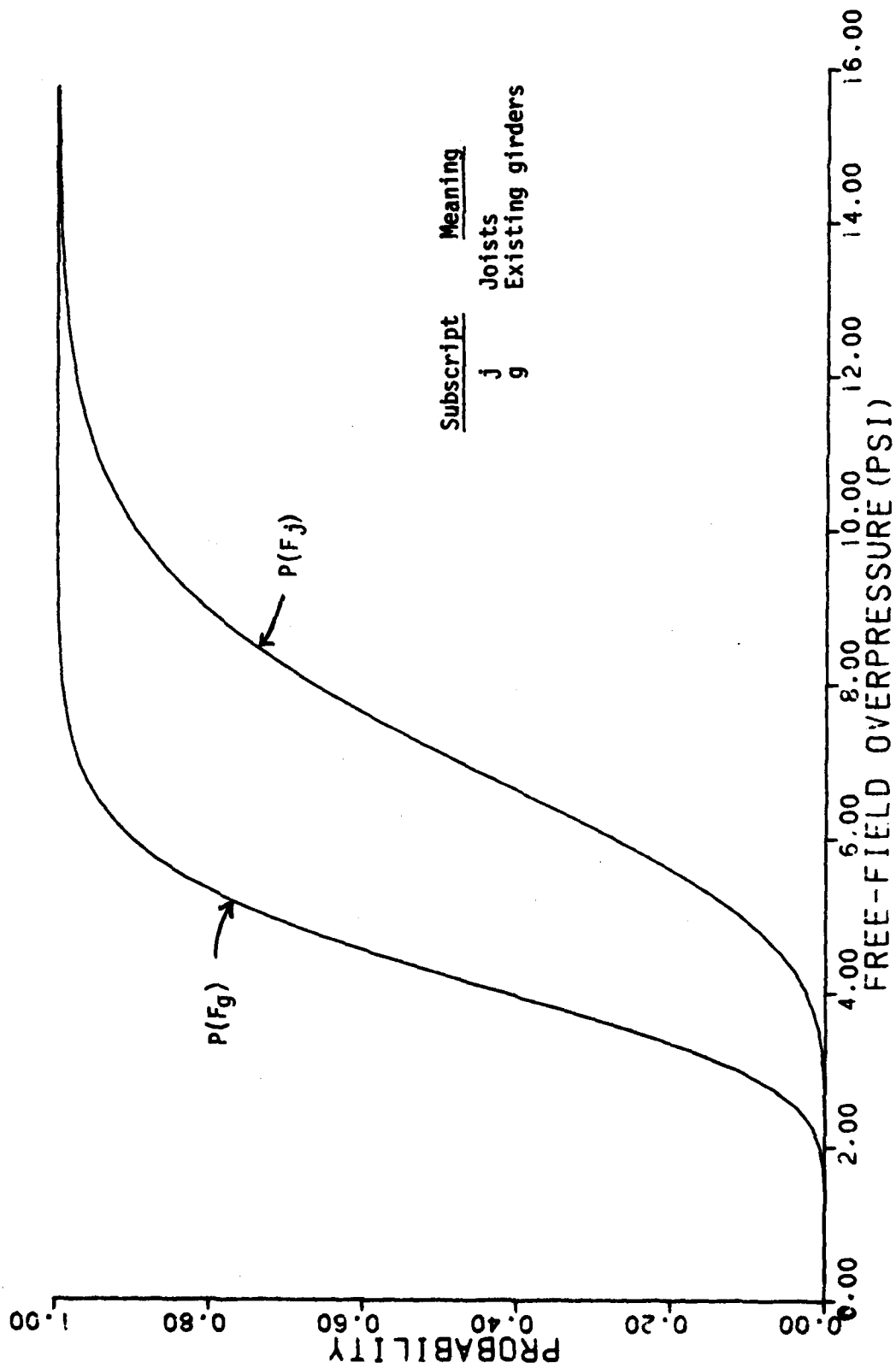


Figure B-20. Joists and existing girders failure probabilities, Scheme 1, Tea Pot House.

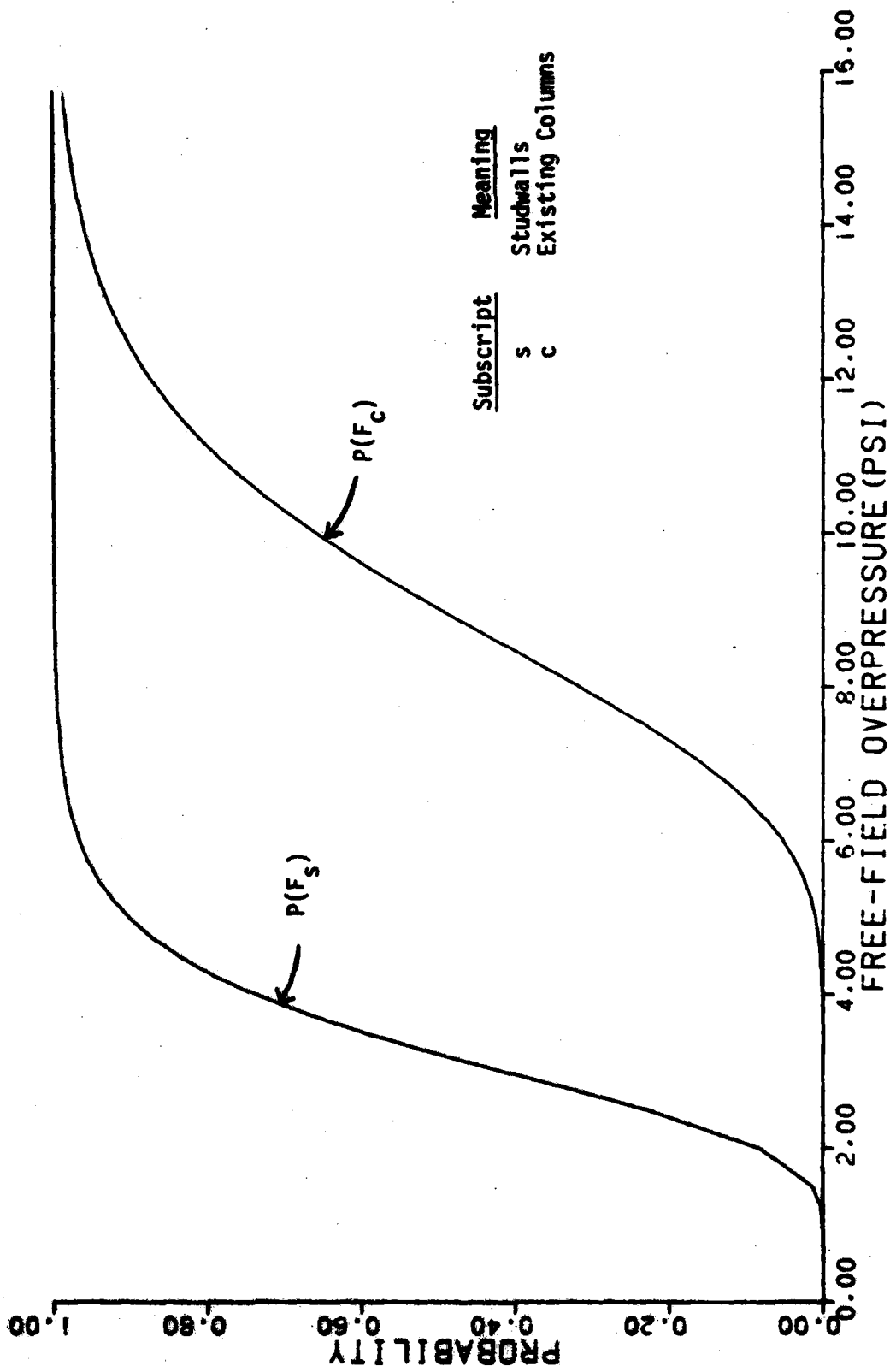


Figure B-21. Studwalls and existing columns failure probabilities, Scheme 1 Tea Pot House.

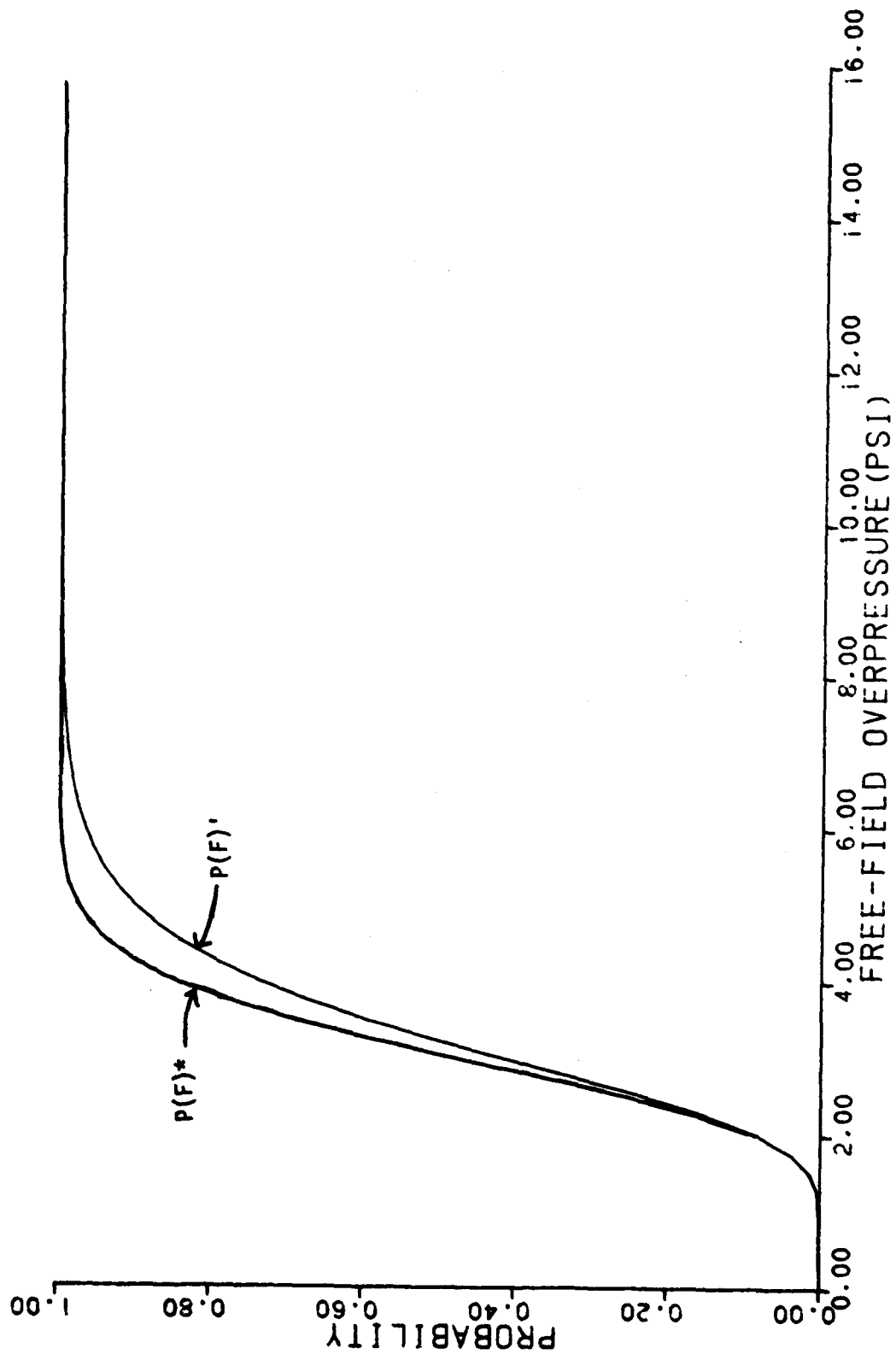


Figure B-22. Probability of floor system failure, upper and lower bound, Scheme 1, Tea Pot House.

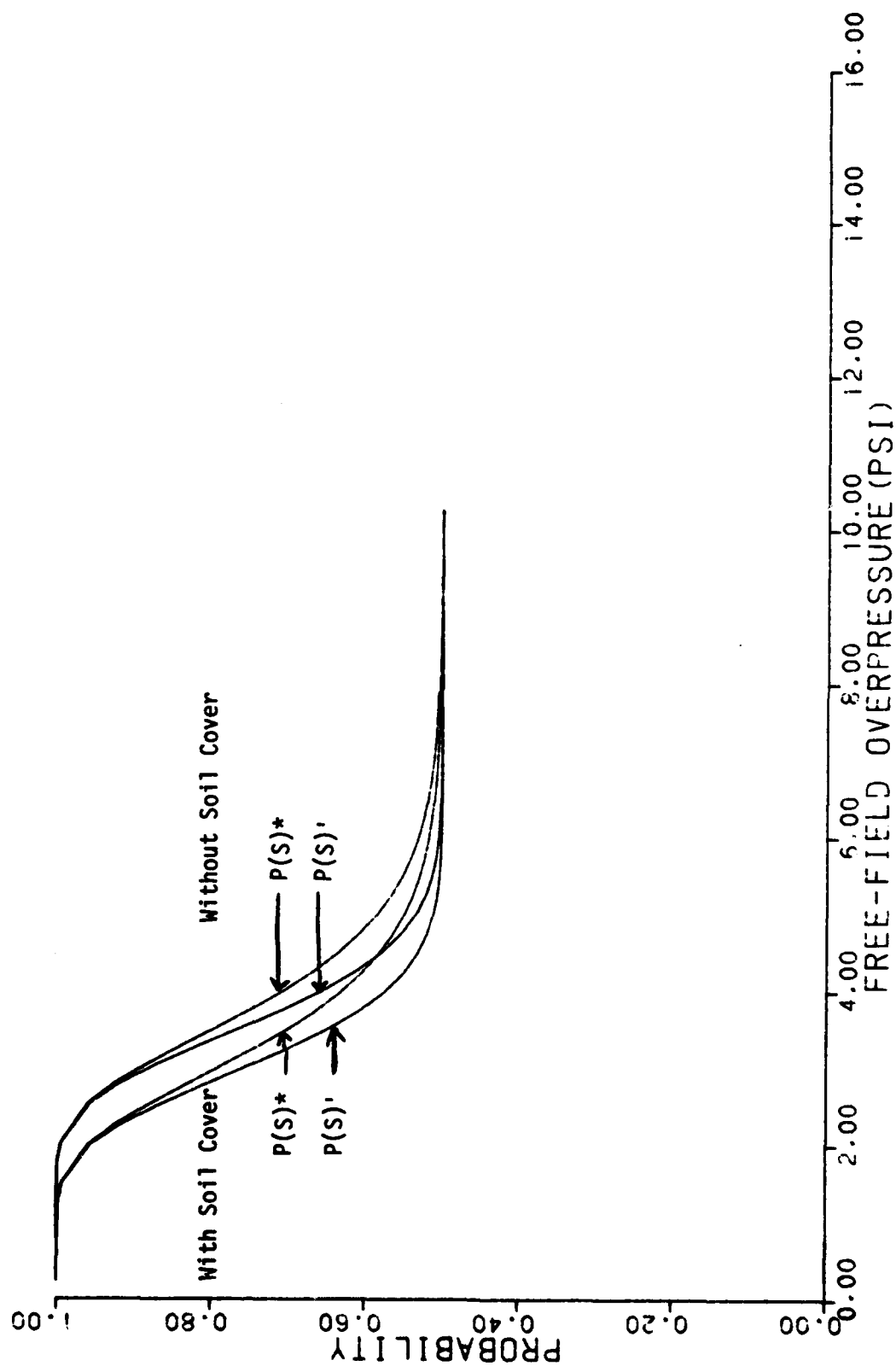


Figure B-23. Probability of people survival, upper and lower bound, Scheme 1, Tea Pot House.

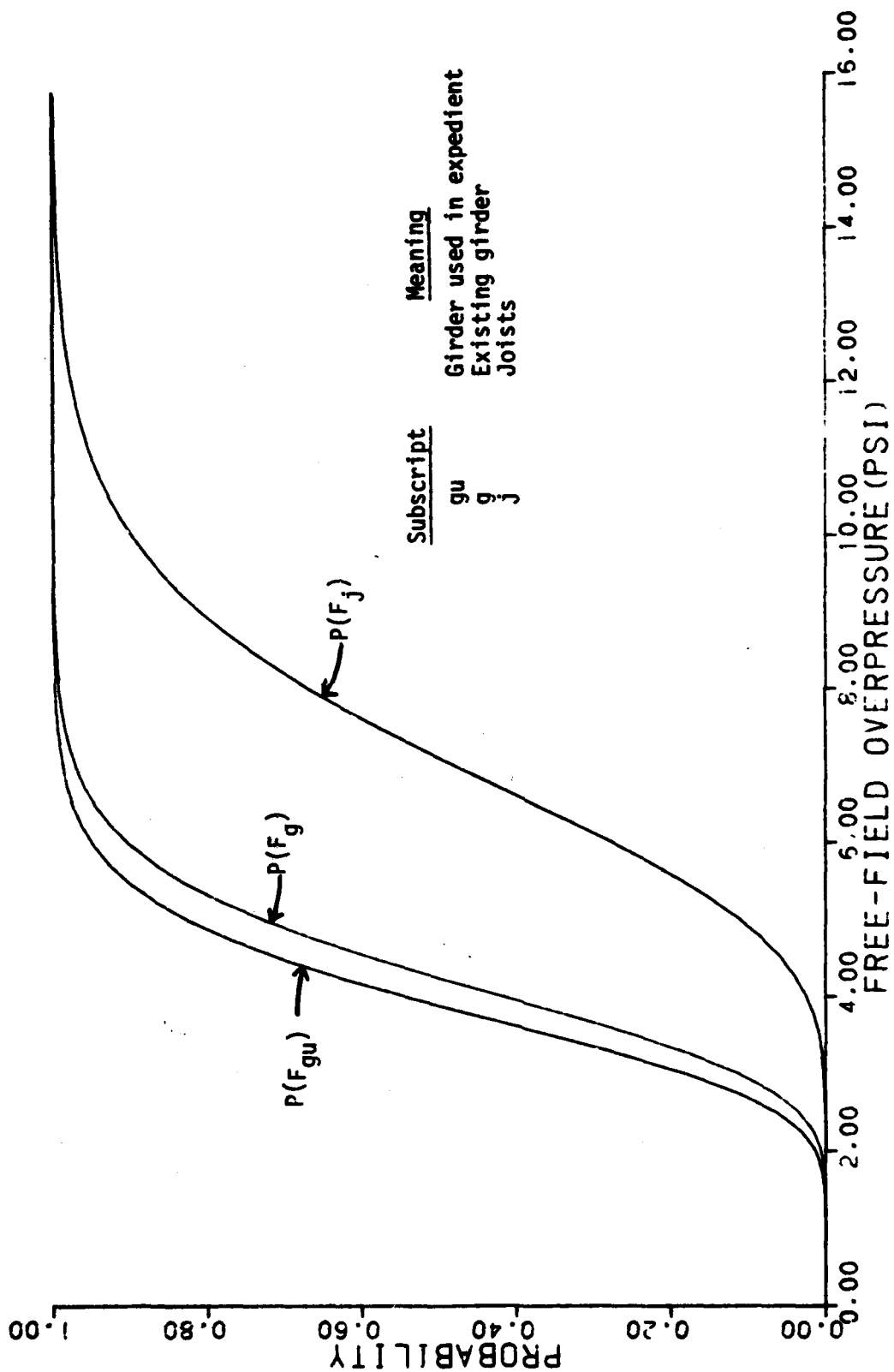


Figure B-24. Joist and girder failure probabilities, Scheme 2, Tea Pot House.

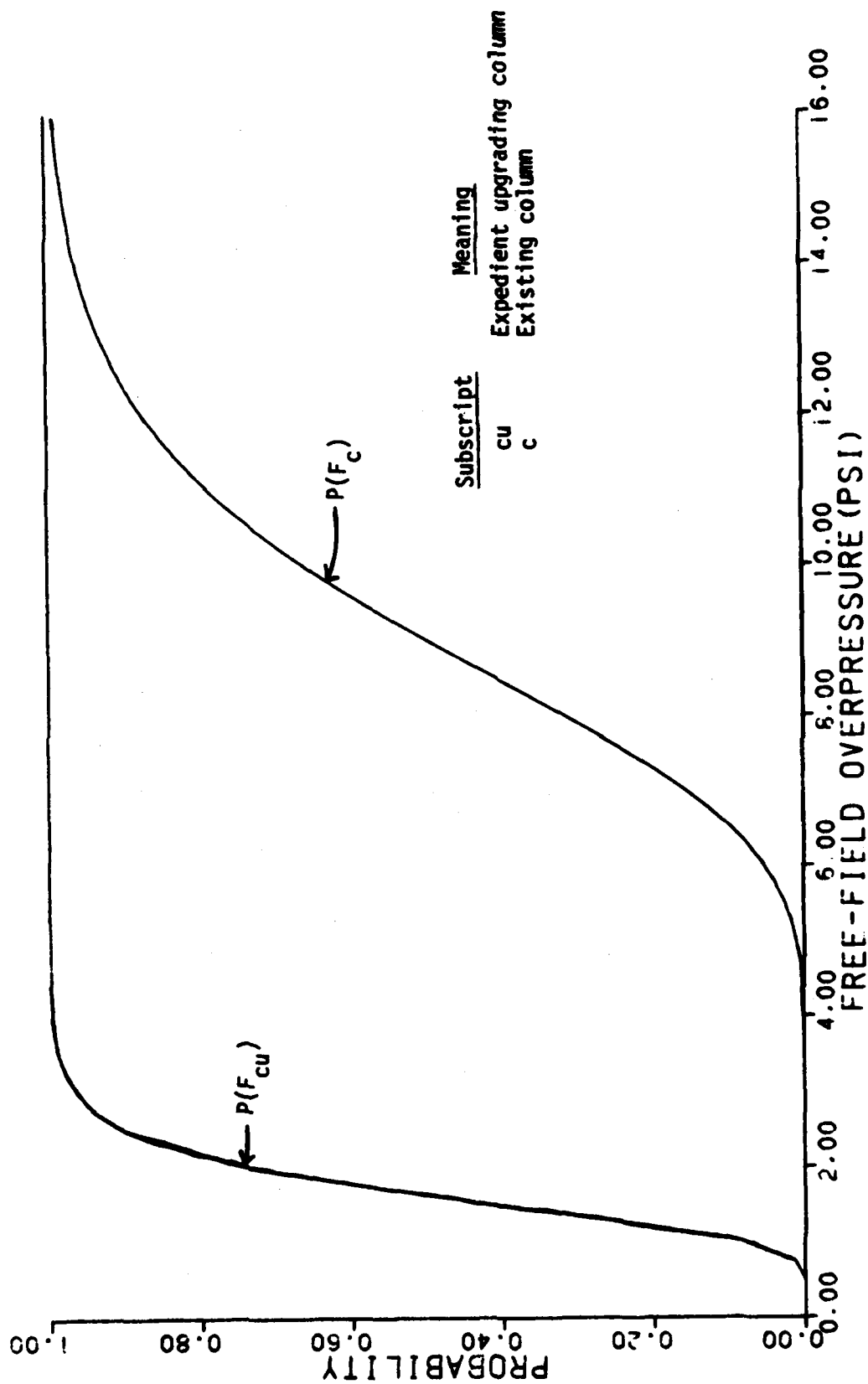


Figure B-25. Failure probabilities of columns, Scheme 2, Tea Pot House.

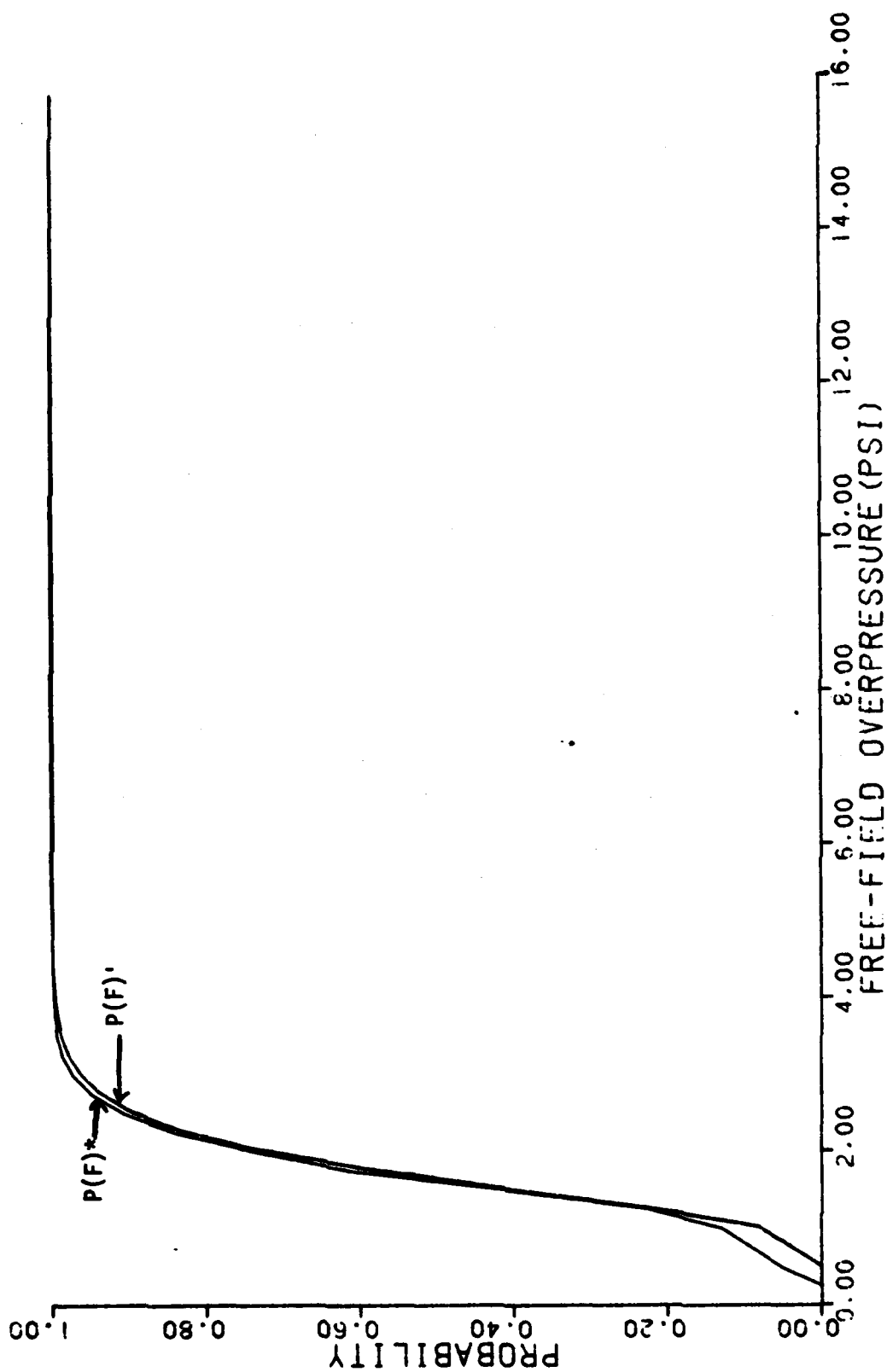


Figure B-26. Probability of floor system failure, upper and lower bound, Scheme 2, Tea Pot House.

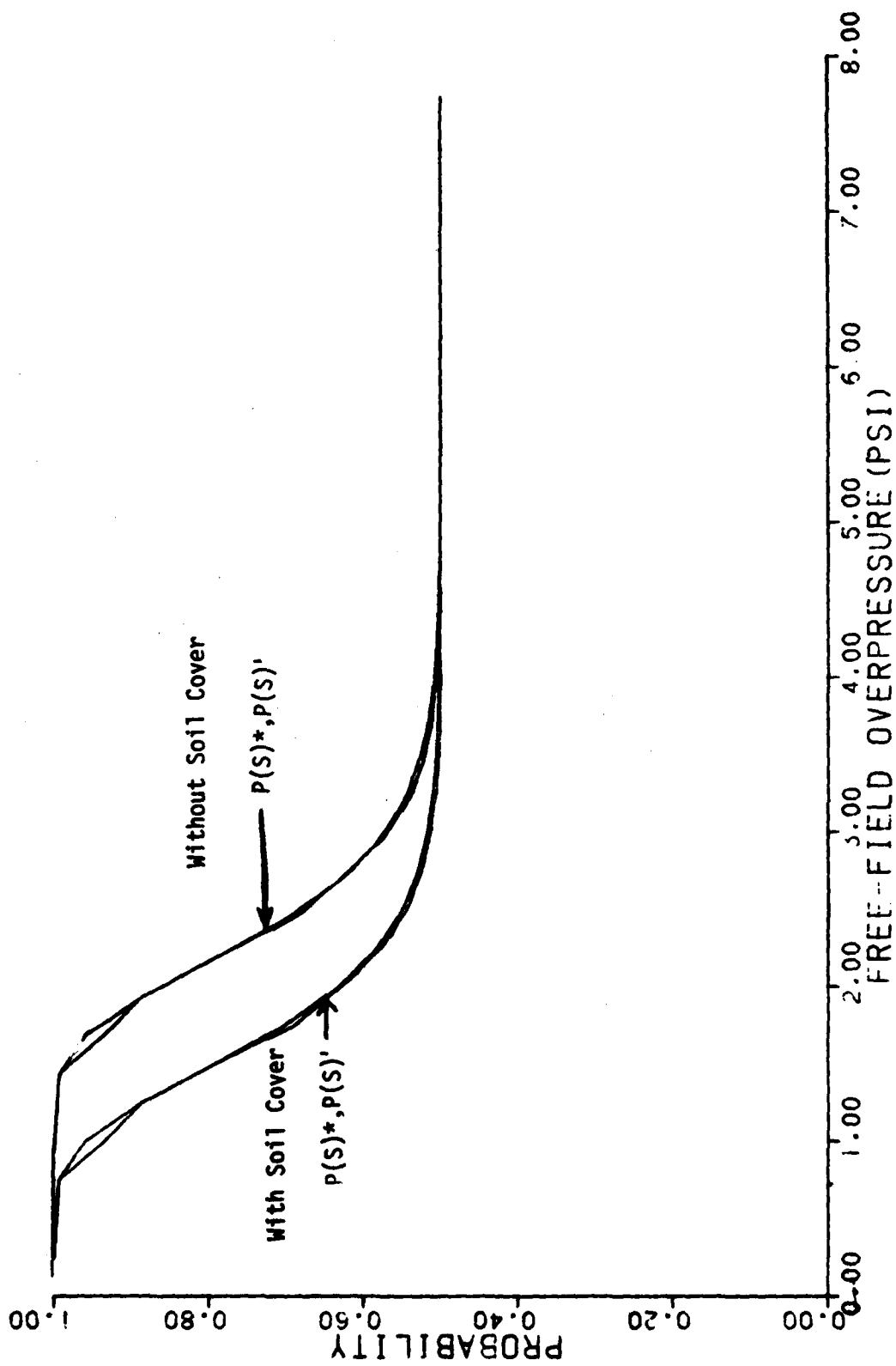


Figure B-27. Probability of people survival, upper and lower bound, Scheme 2, Tea Pot House.

APPENDIX C

STRUCTURAL FAILURE AND PEOPLE SURVIVAL PROBABILITY DATA

This appendix contains detailed results on the probability of structural failure and the probability of people survival for the reinforced concrete shelters described in Chapter 4. The general concept of the basic basement shelters is illustrated in Figure 1. These basements were designed for live loads in the range from 50 psi to 250 psi and span lengths from 12 ft to 20 ft. The basic design data are given in Table 2. Each of the 12 slabs was analyzed as upgraded using four expedient upgrading schemes illustrated in Figure 9. This resulted in 60 sets of shelters whose analysis data are included in Table 3. Results included here are upper and lower bounds on the probability of slab failure and upper and lower bounds on the probability of people survival.

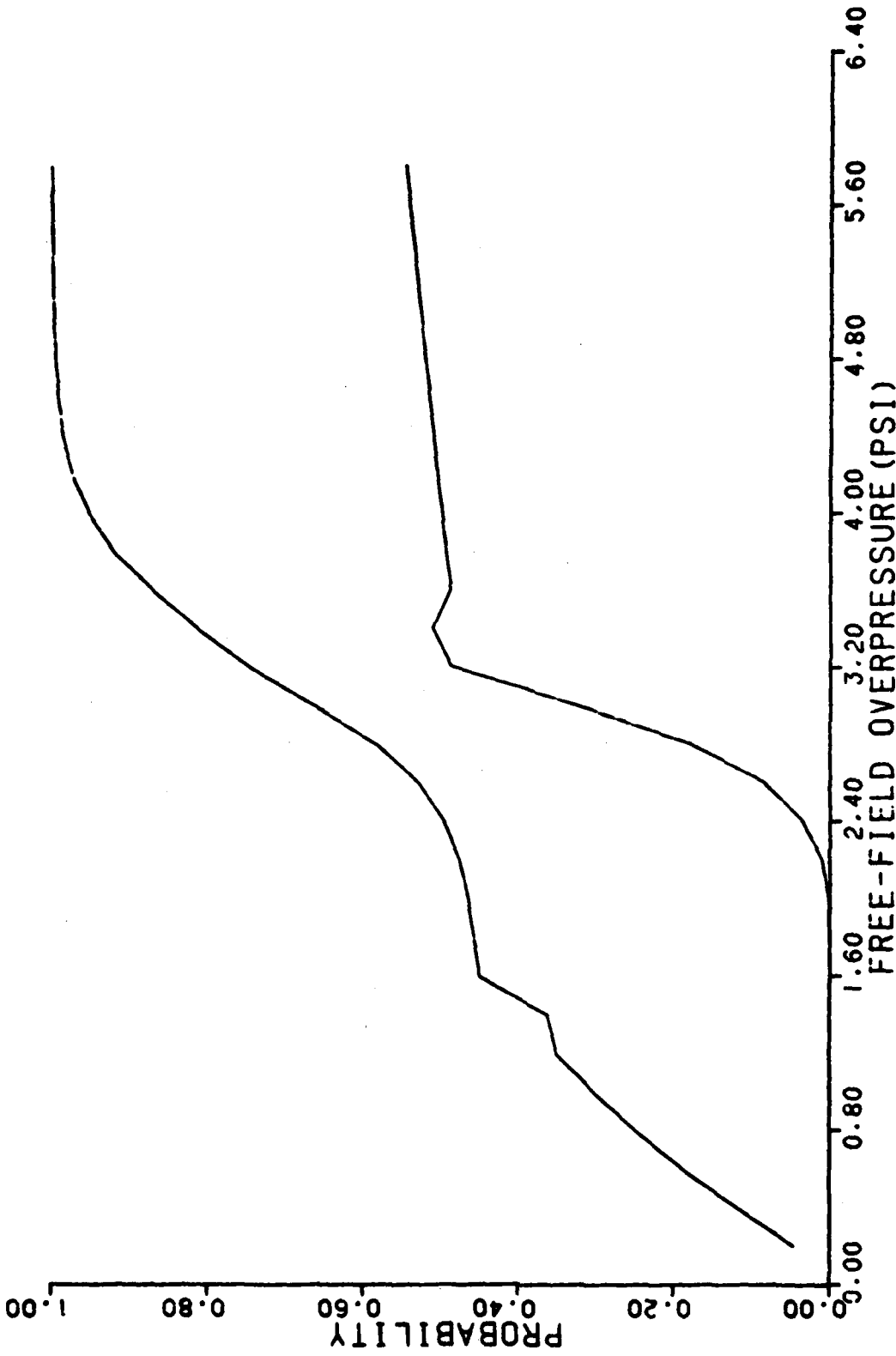


Figure C-1. Probability of slab failure (upper and lower bounds) case 1A.

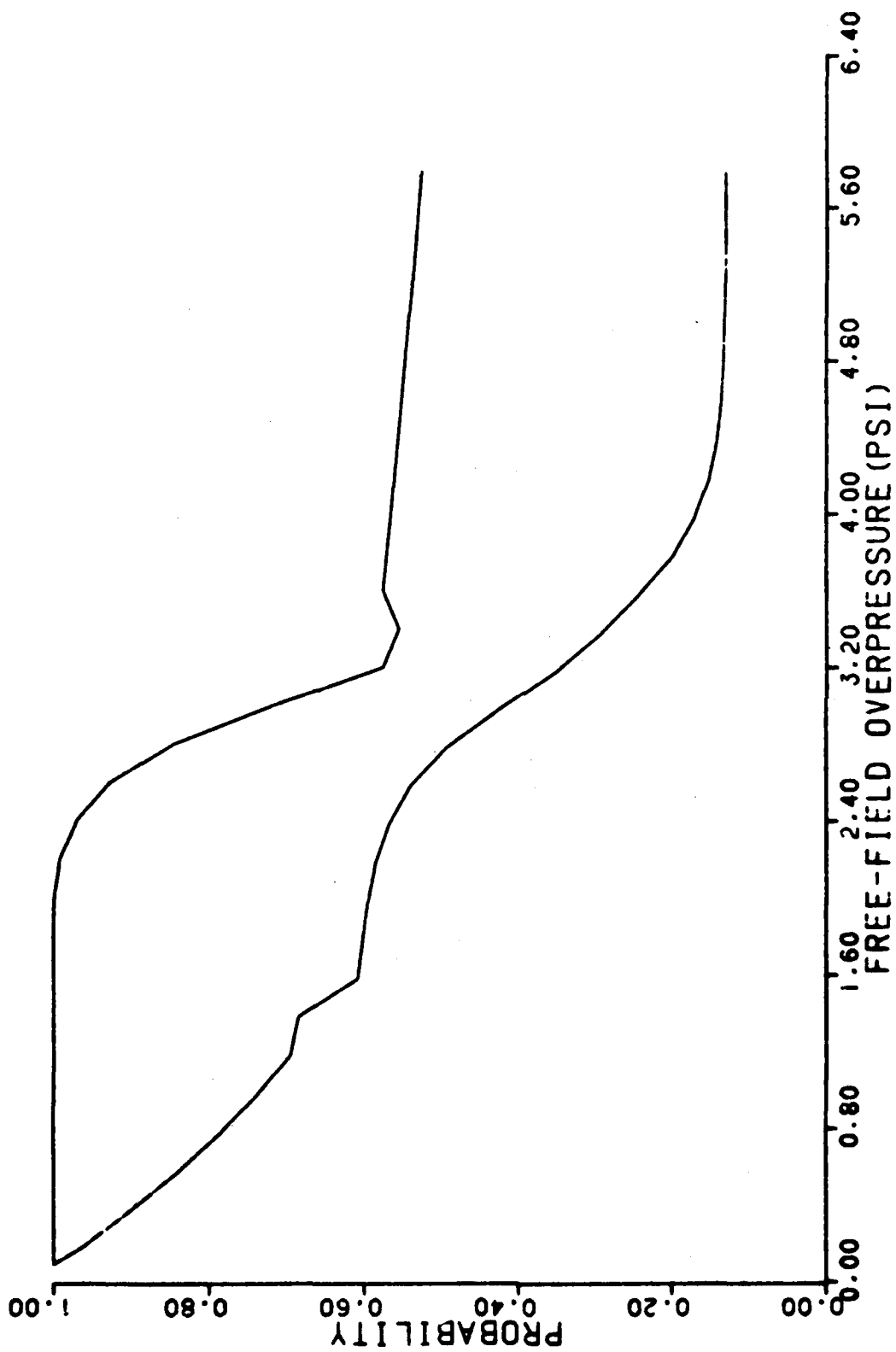


Figure C-2. Probability of people survival (upper and lower bounds) case 1A.

CASE 1B2

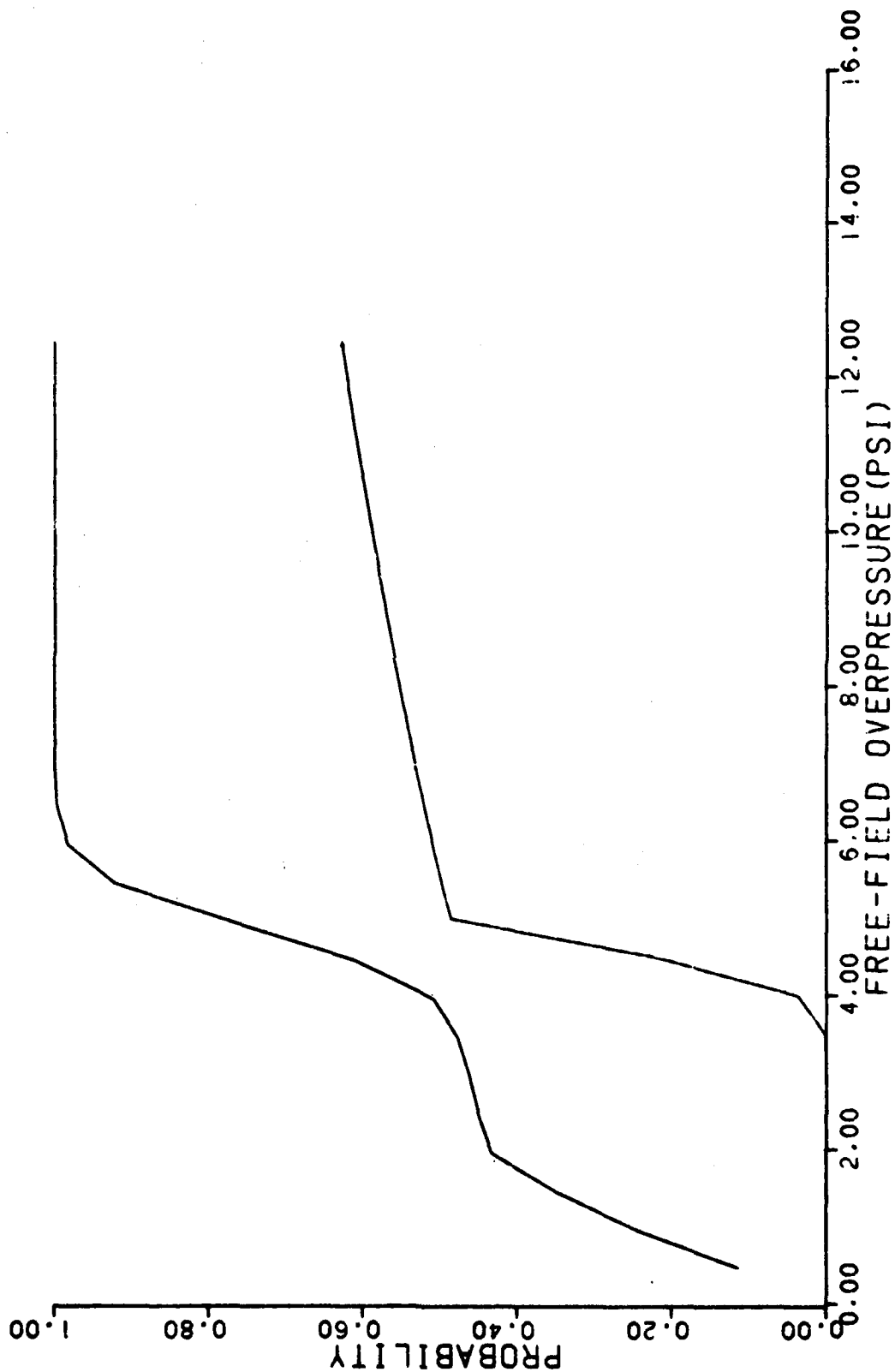


Figure C-3. Probability of slab failure (upper and lower bounds) case 1B.

CASE 1B3

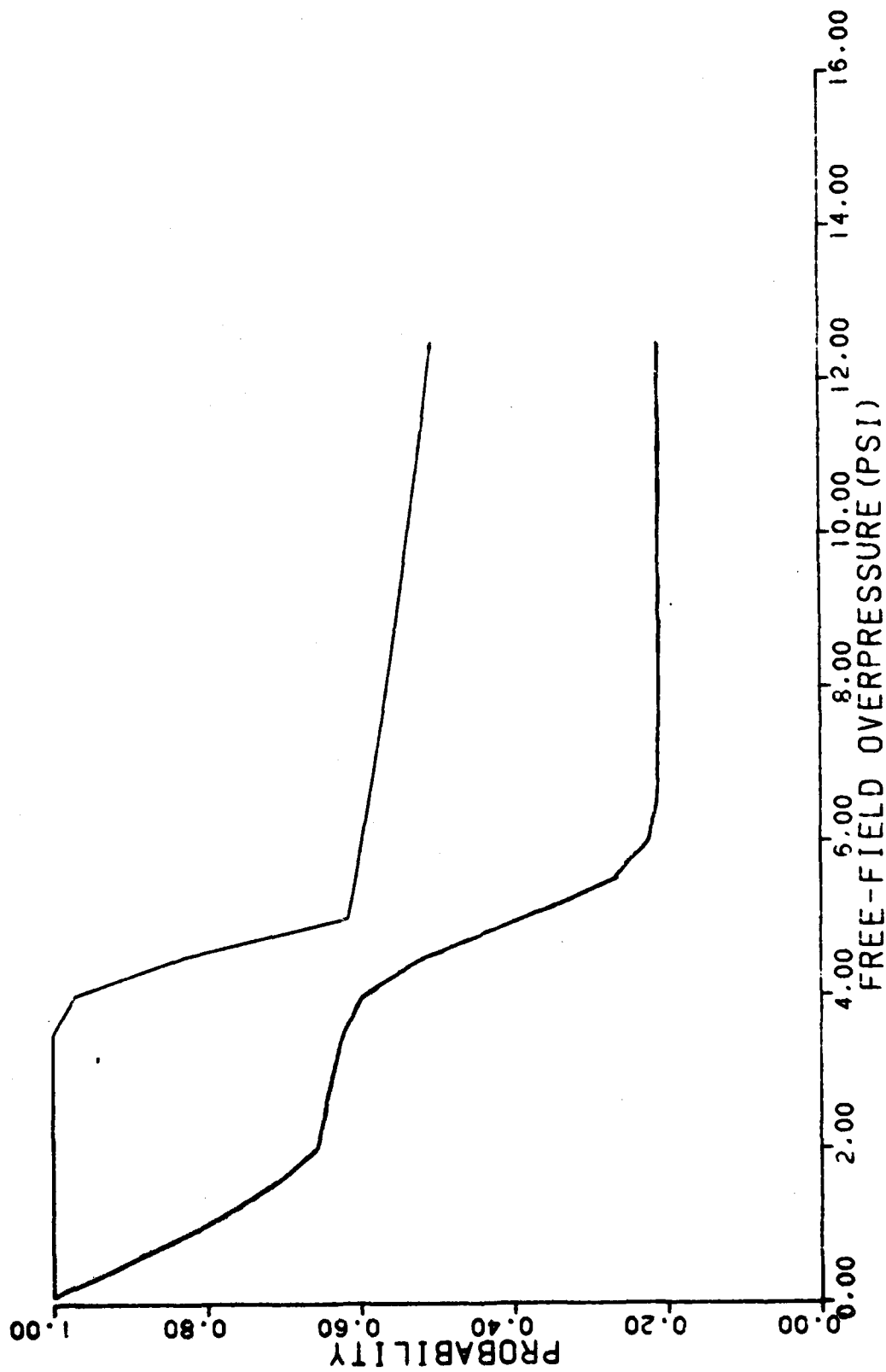


Figure C-4. Probability of people survival (upper and lower bounds) case 1B.

CASE 1C2

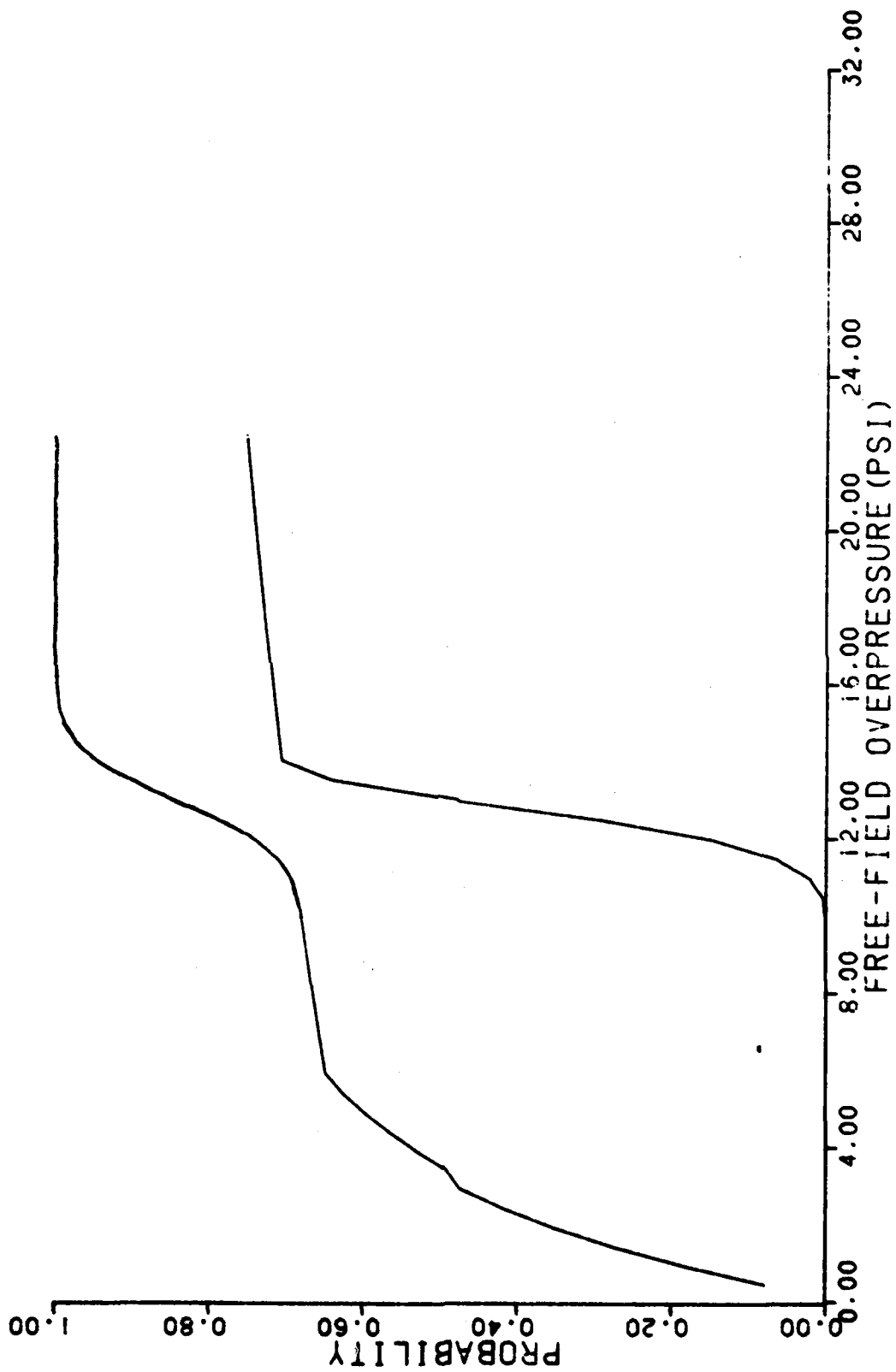


Figure C-5. Probability of slab failure (upper and lower bounds) case 1C.

CASE 1C3

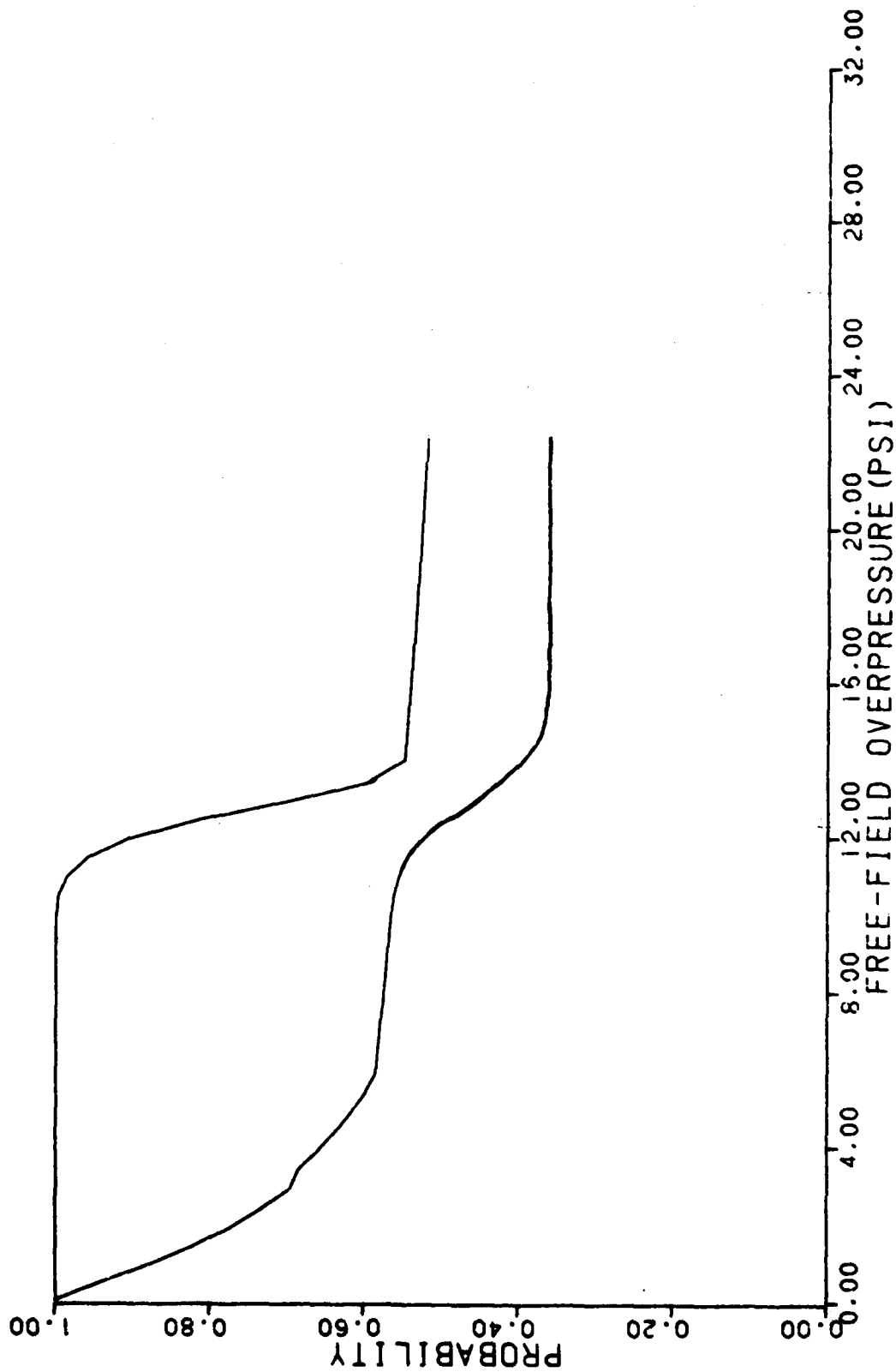


Figure C-6. Probability of people survival (upper and lower bounds) case 1C.

CASE 1D2



Figure C-7. Probability of slab failure (upper and lower bounds) case 1D.

CASE 1D3

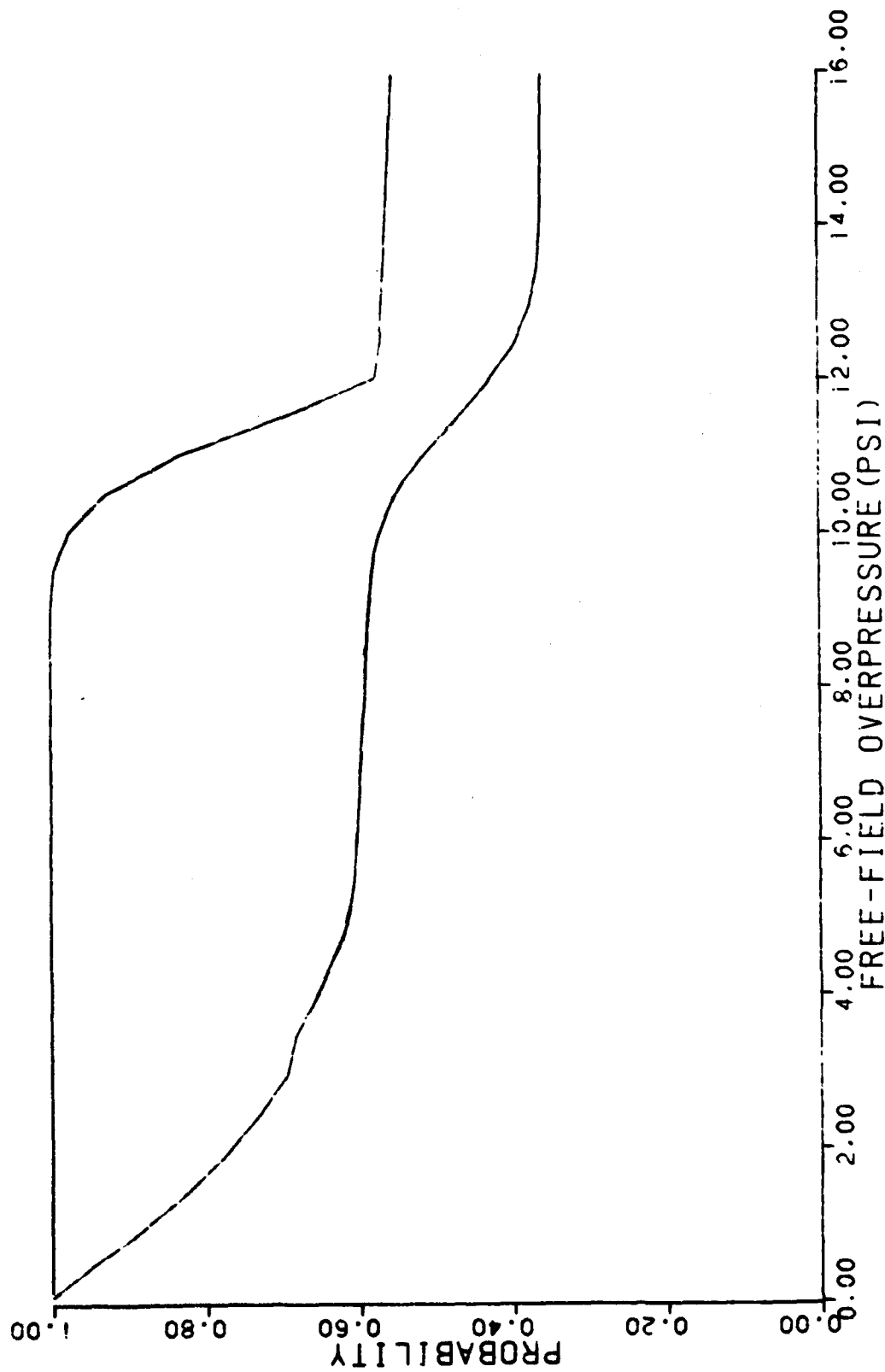


Figure C-8. Probability of people survival (upper and lower bounds) case 1D.

CASE 1E2

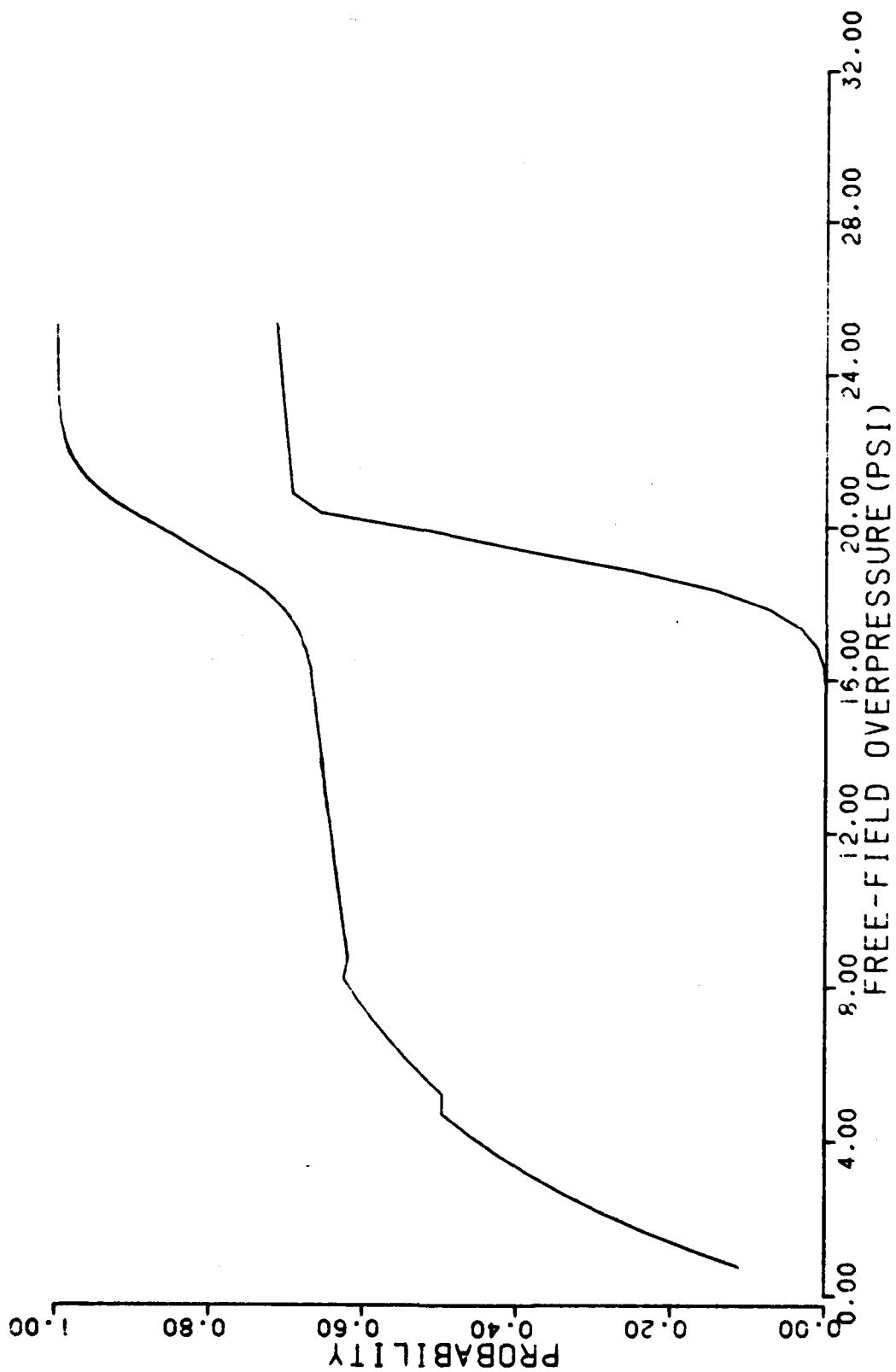


Figure C-9. Probability of slab failure (upper and lower bounds) case 1E.

CASE 1E3

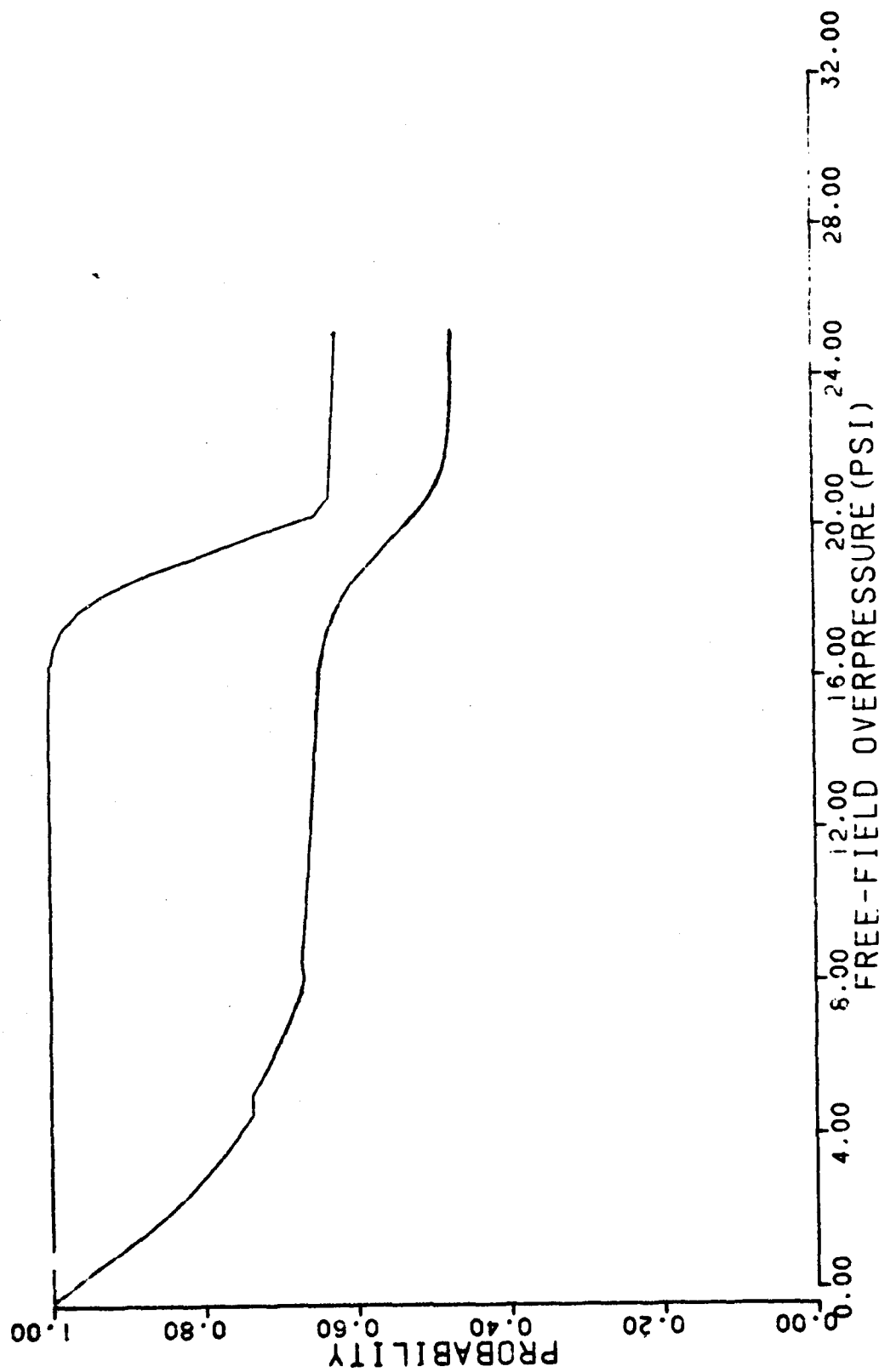


Figure C-10. Probability of people survival (upper and lower bounds) case 1E.

CASE 2A2



Figure C-11. Probability of slab failure (upper and lower bounds) case 2A.

CASE 2A3

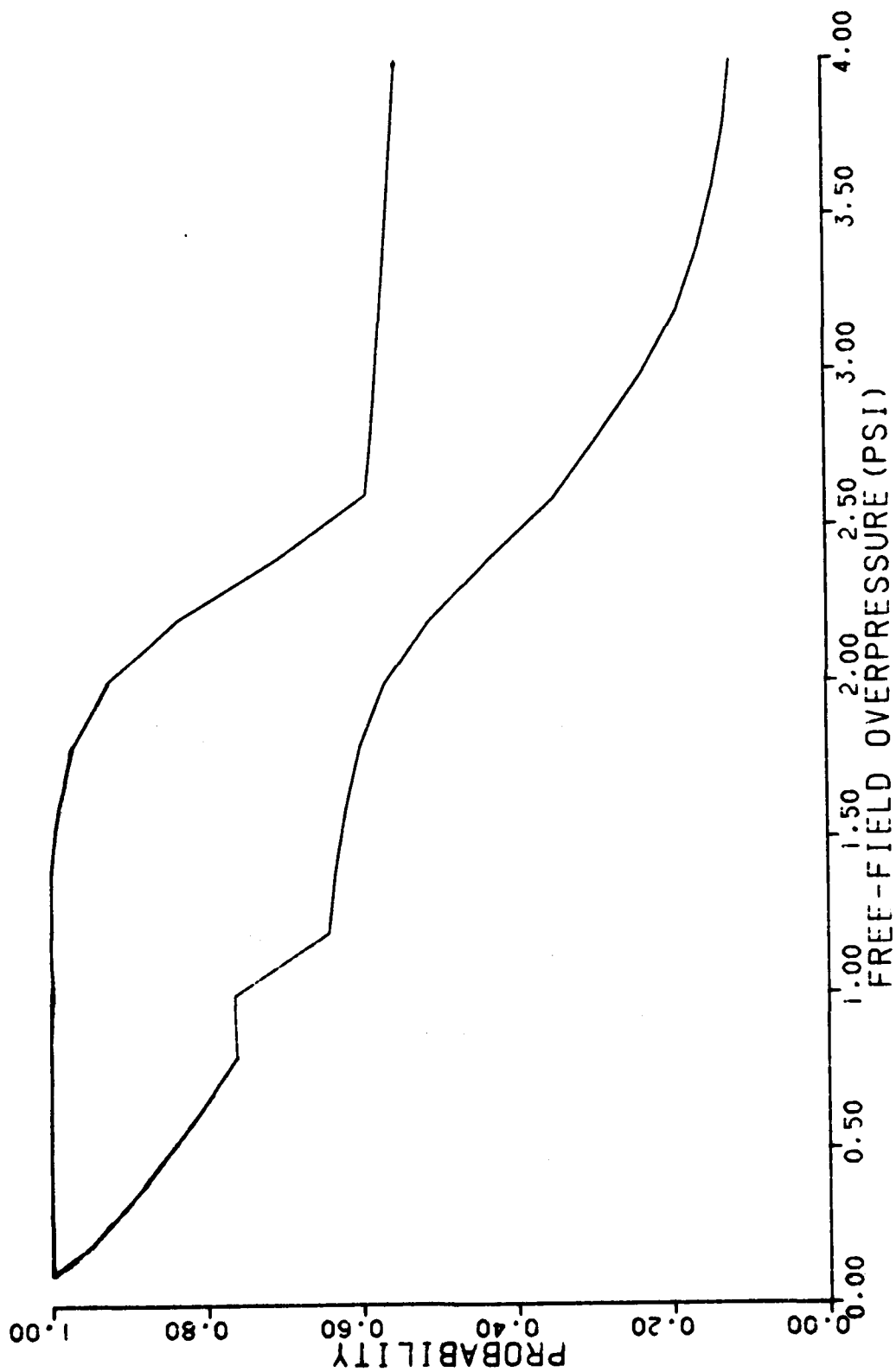


Figure C-12. Probability of people survival (upper and lower bounds) case 2A.

CASE 2B2

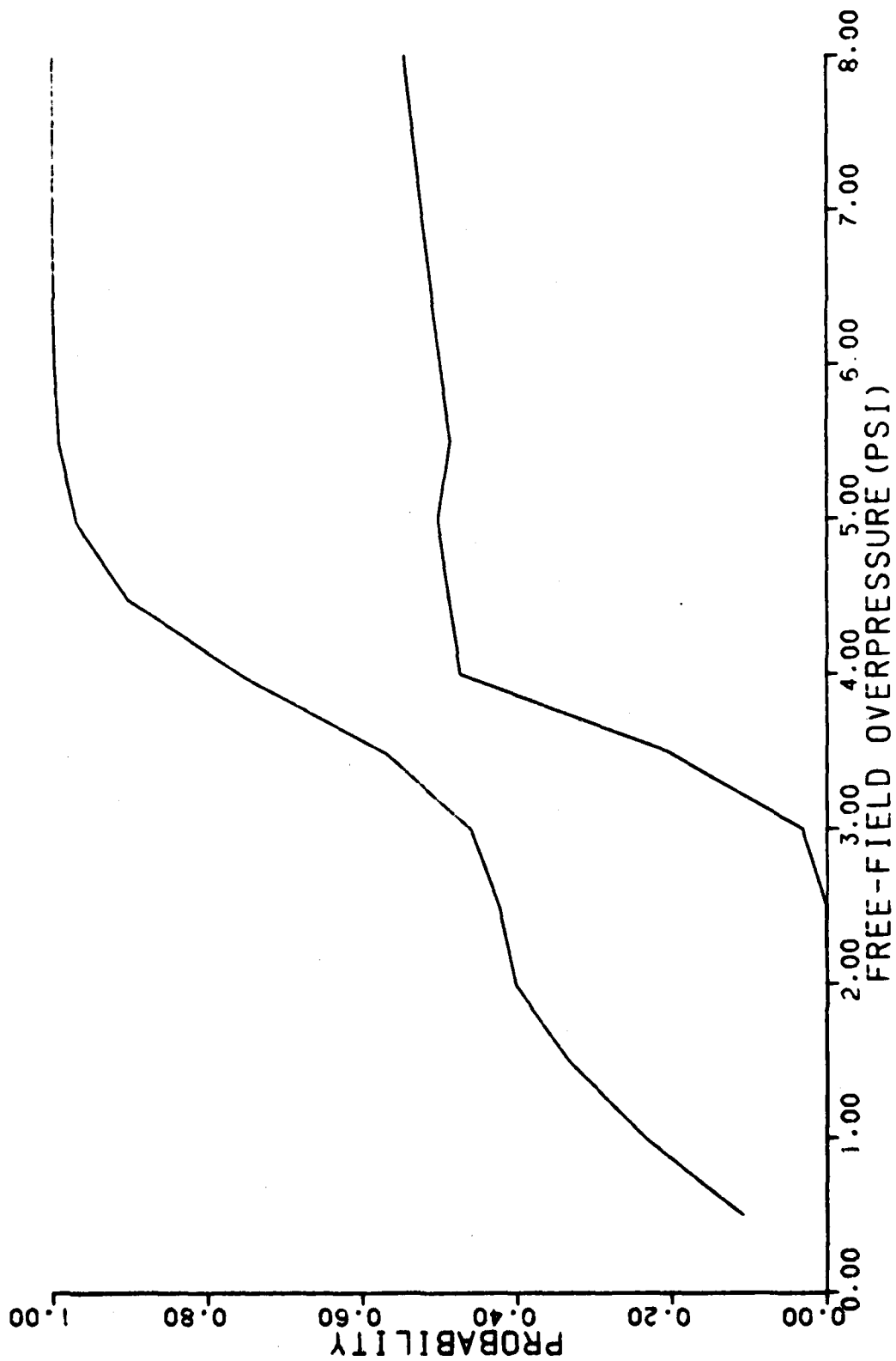


Figure C-13. Probability of slab failure (upper and lower bounds) case 2B.

CASE 2B3

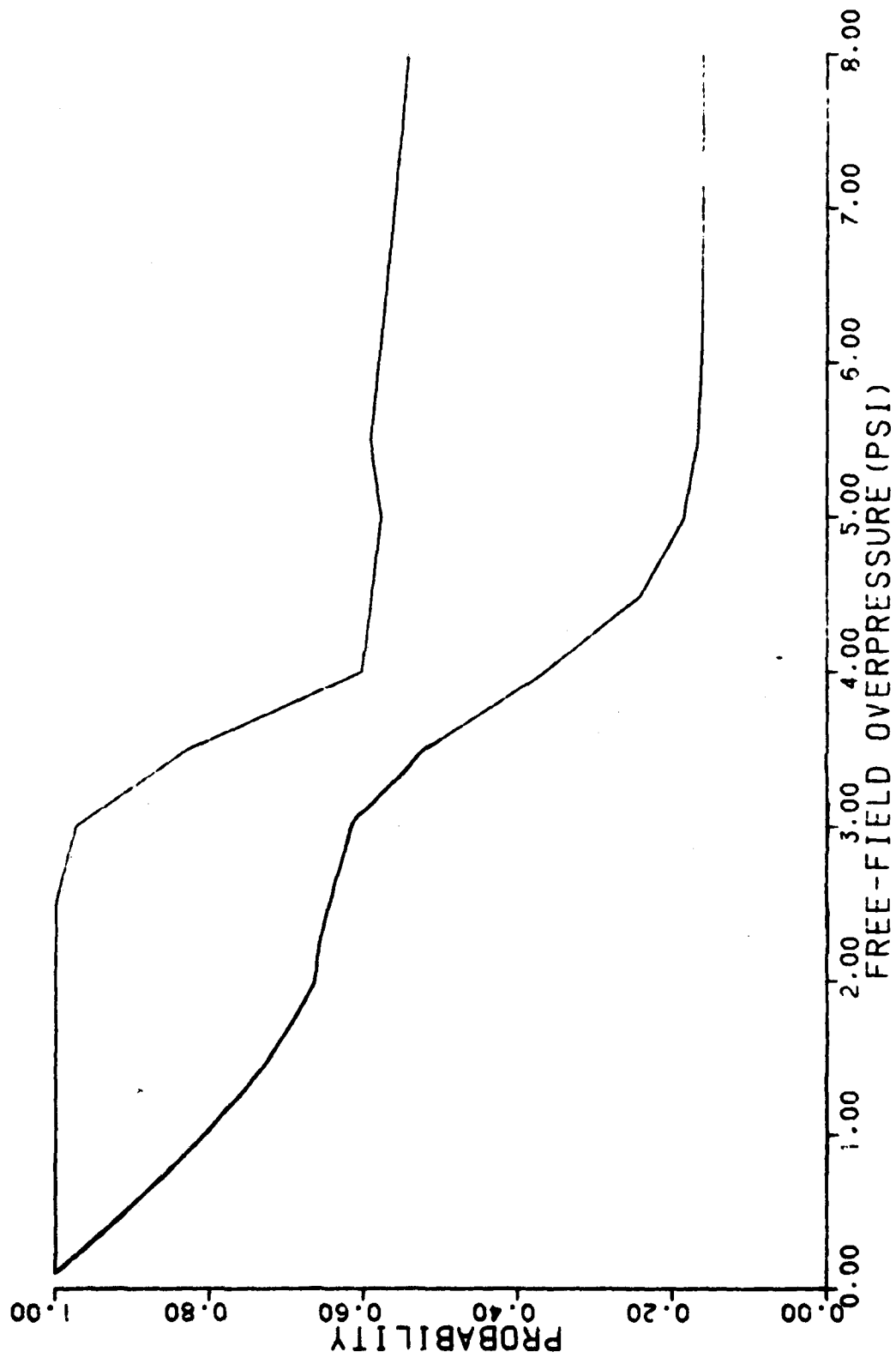


Figure C-14. Probability of people survival (upper and lower bounds) case 2B.

CASE 2C2



Figure C-15. Probability of slab failure (upper and lower bounds) case 2C.

CASE 203

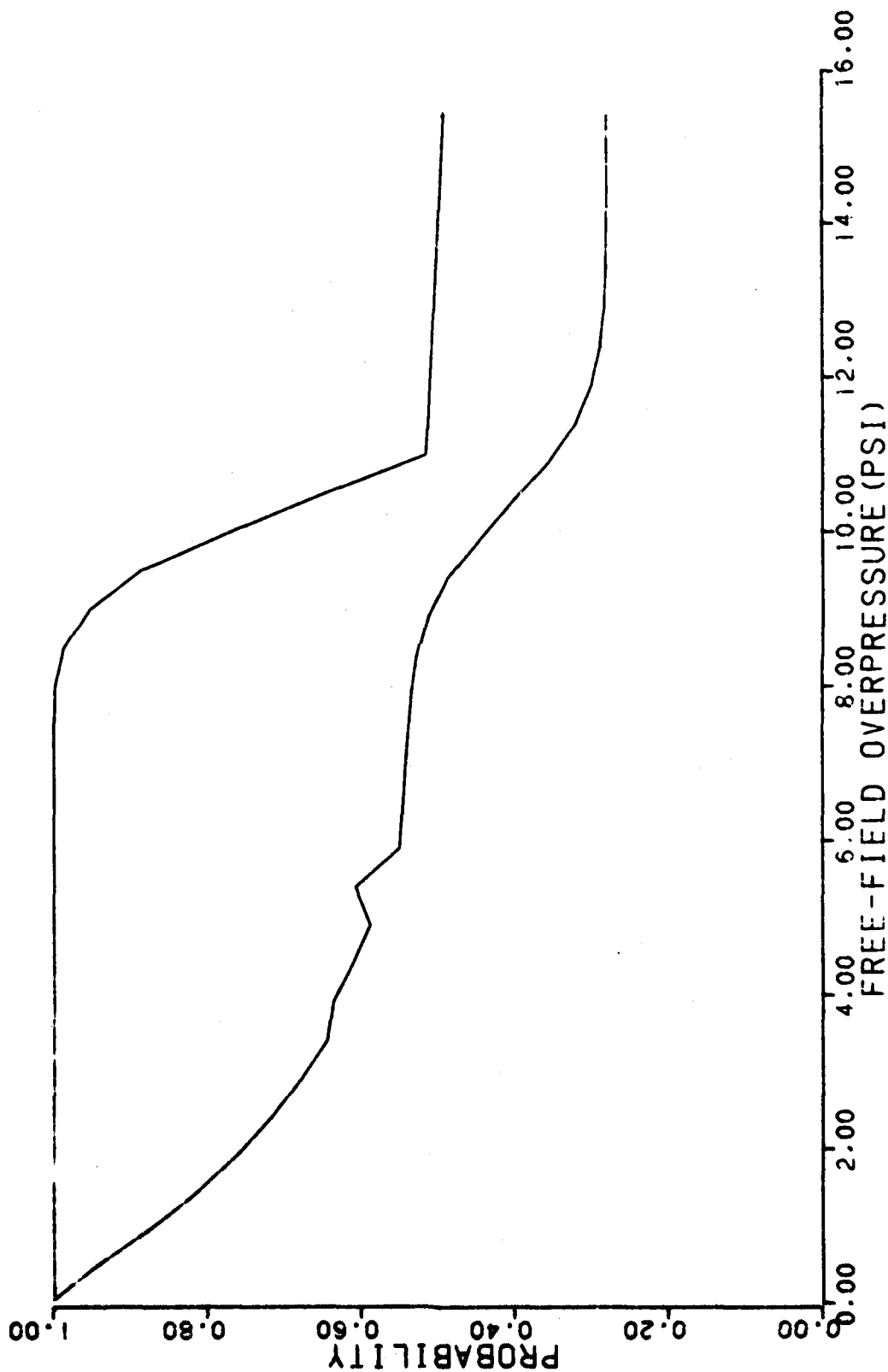


Figure C-16. Probability of people survival (upper and lower bounds) case 20.

CASE 2D2



Figure C-17. Probability of slab failure (upper and lower bounds) case 2D.

CASE 2D3

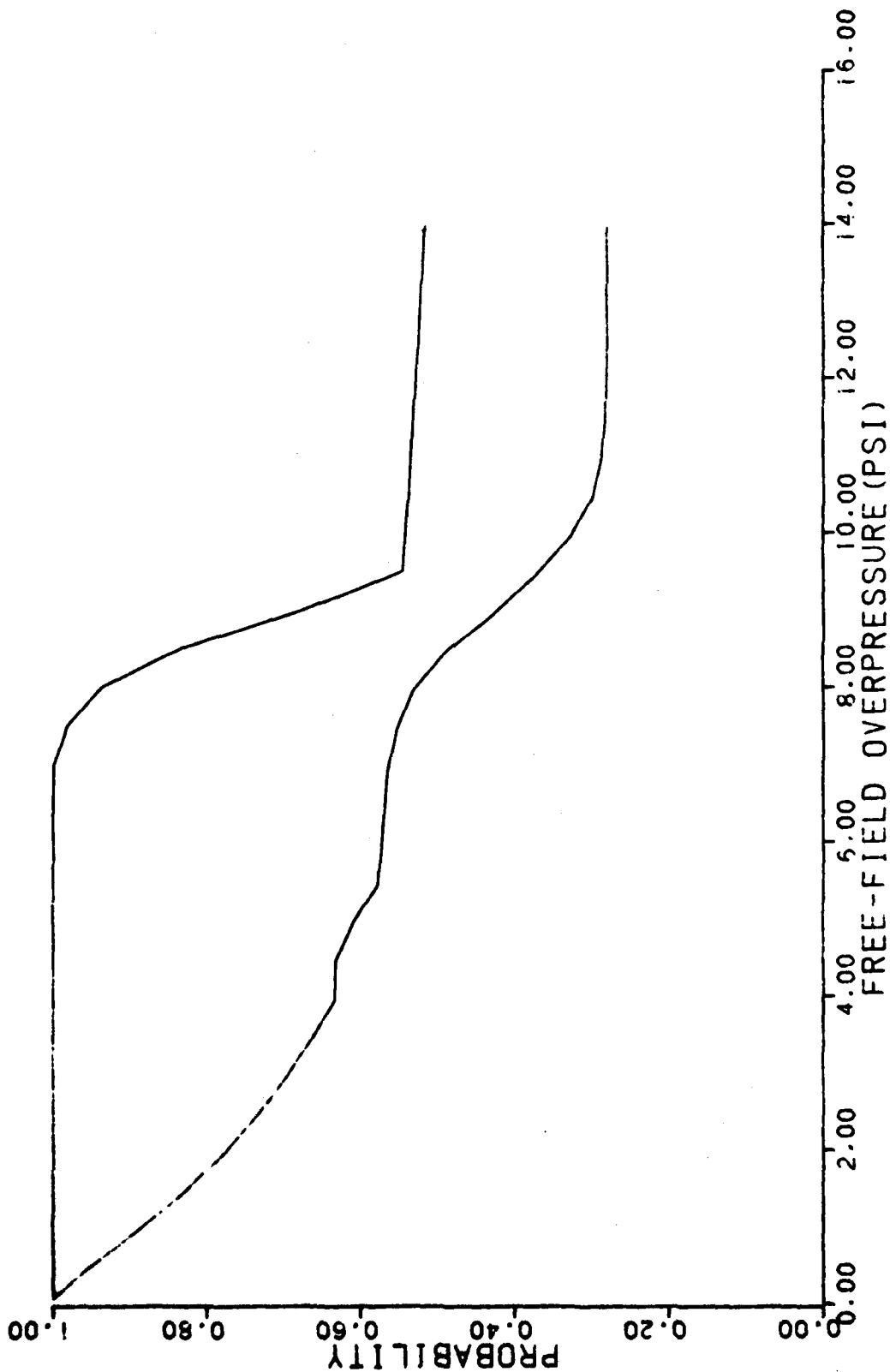


Figure C-18. Probability of people survival (upper and lower bounds) case 2D.

CASE 2E2

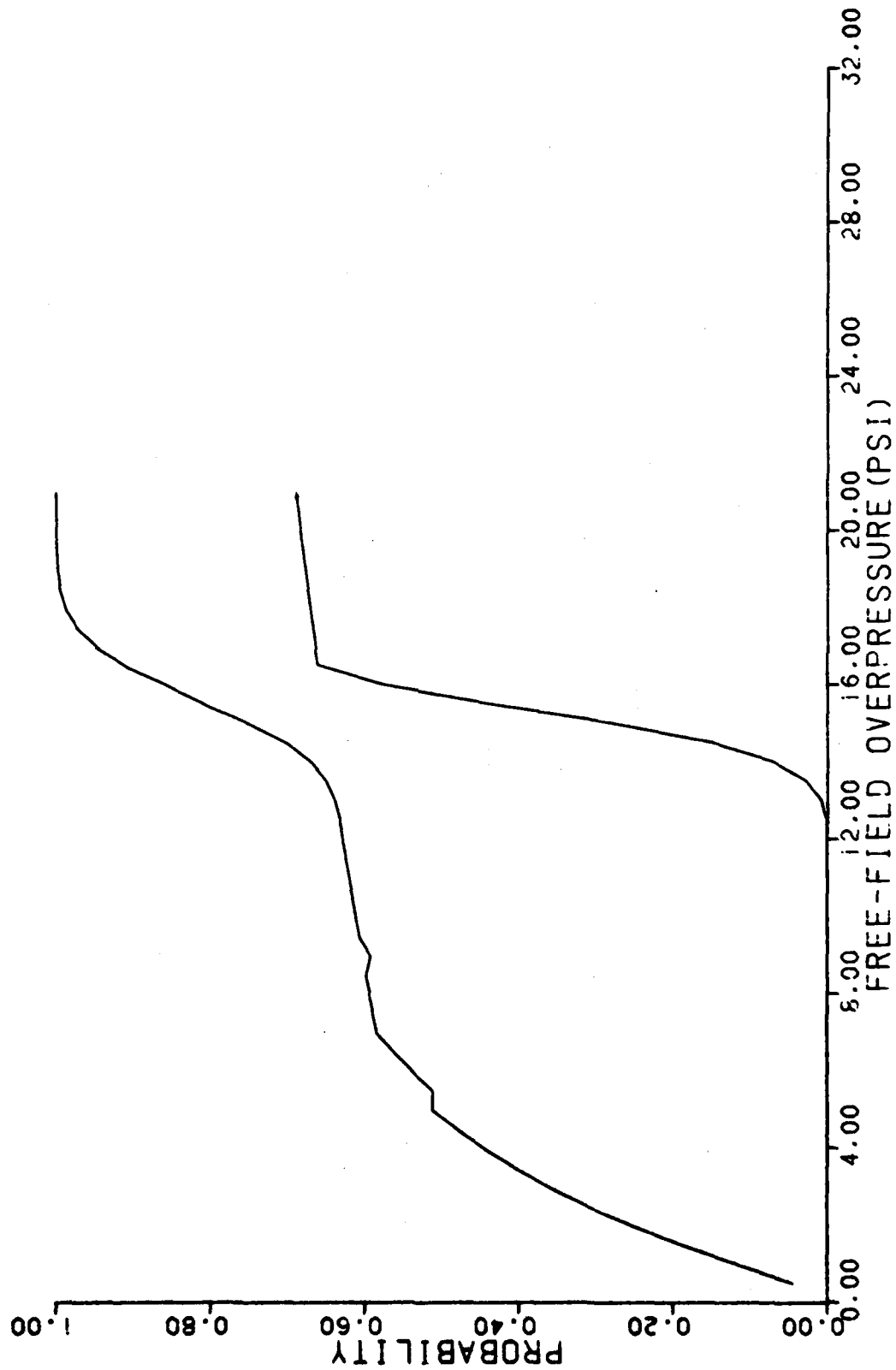


Figure C-19. Probability of slab failure (upper and lower bounds) case 2E.

CASE 2E3

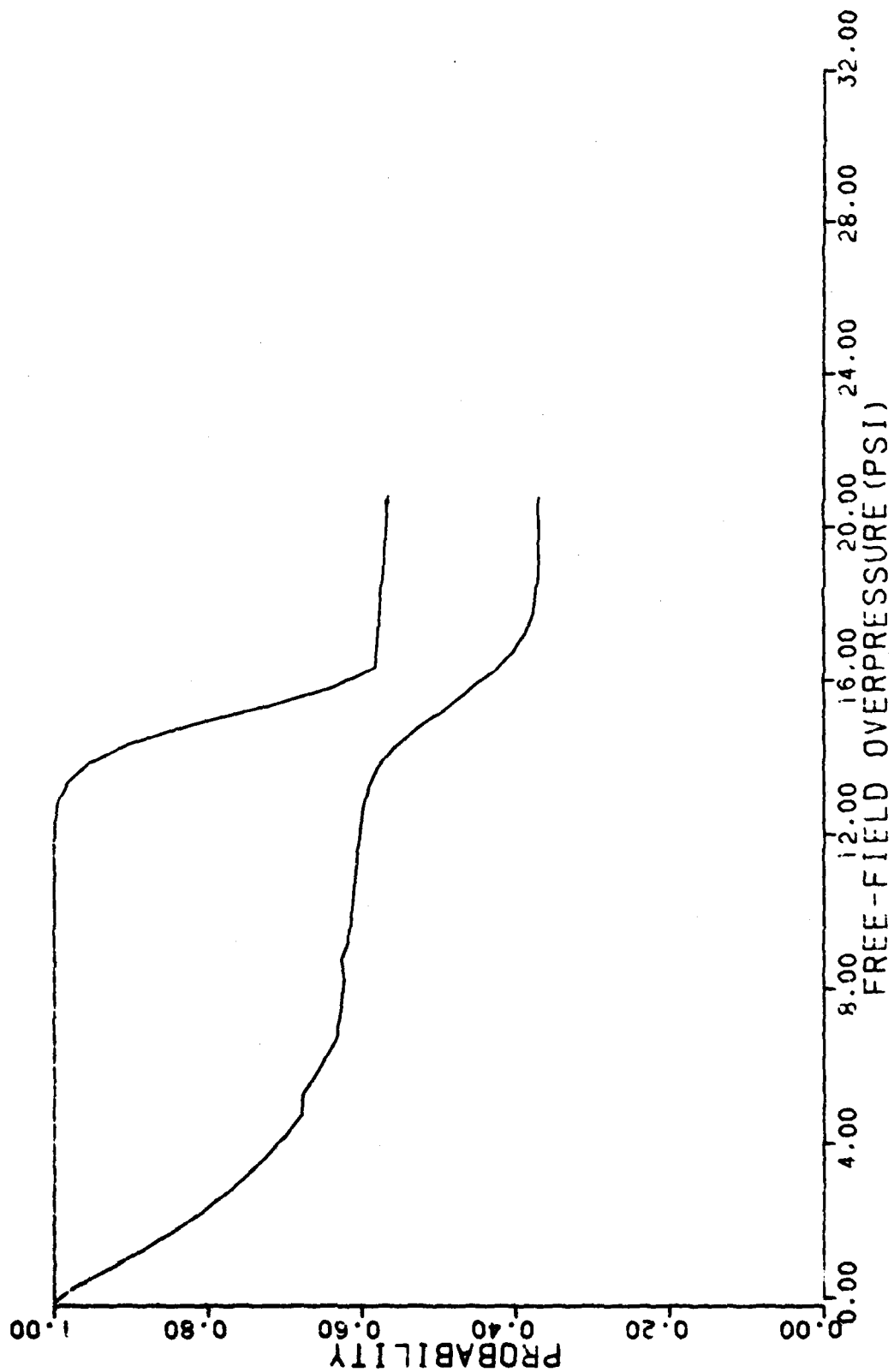


Figure C-20. Probability of people survival (upper and lower bounds) case 2E.

CASE 3A2

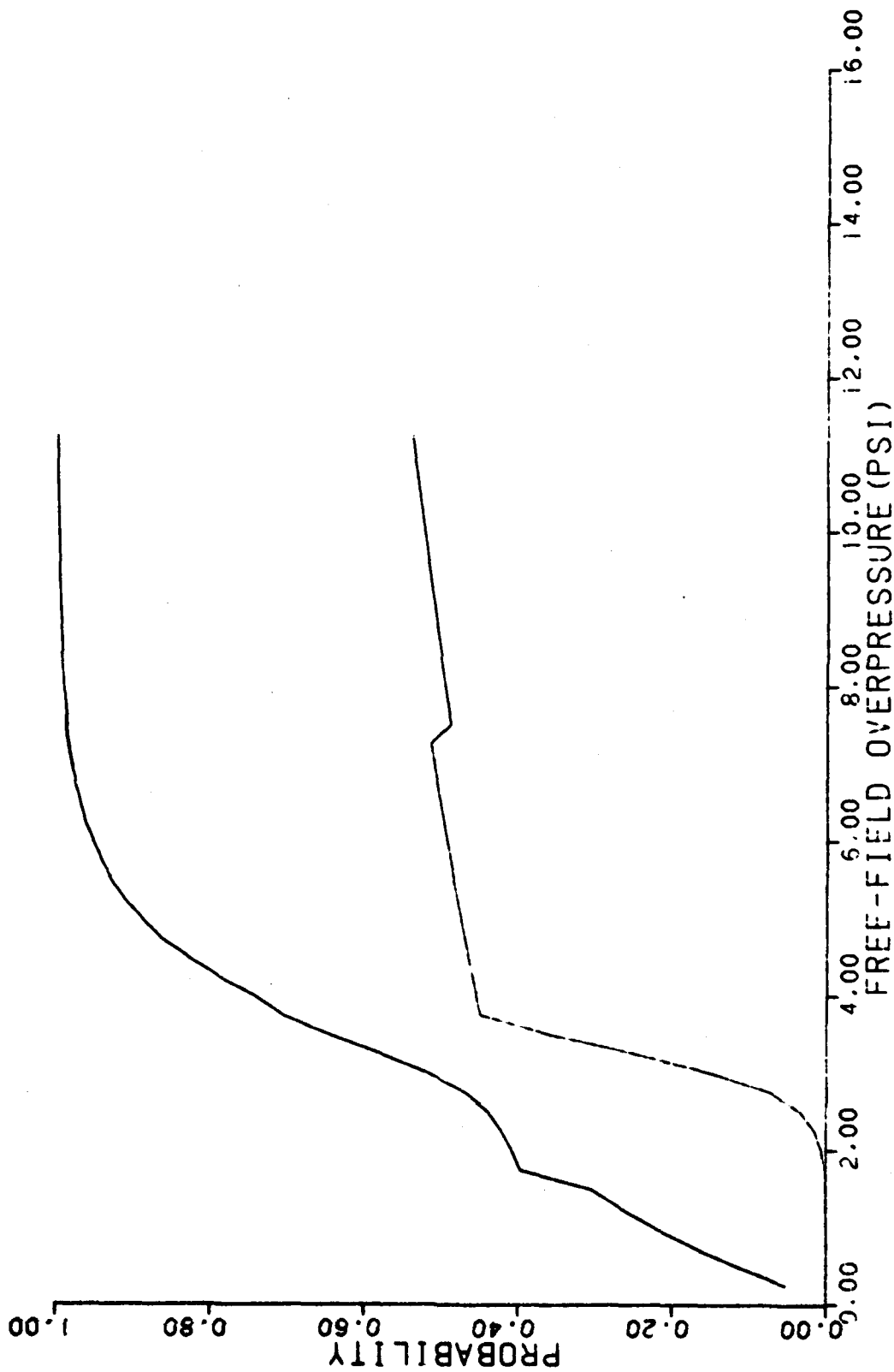


Figure C-21. Probability of slab failure (upper and lower bounds) case 3A.

CASE 3A3

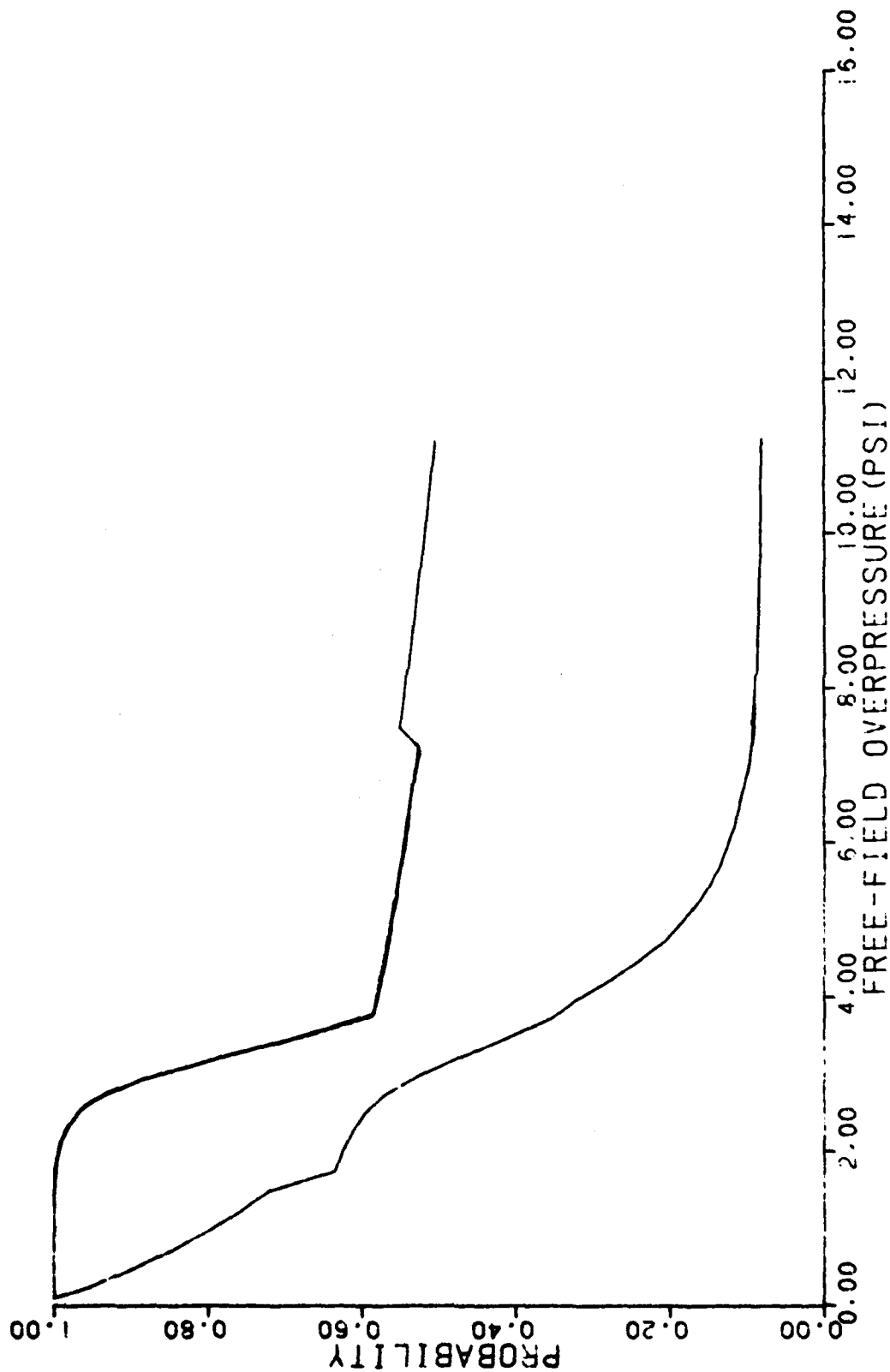


Figure C-22. Probability of people survival (upper and lower bounds) case 3A.

CASE 3B2

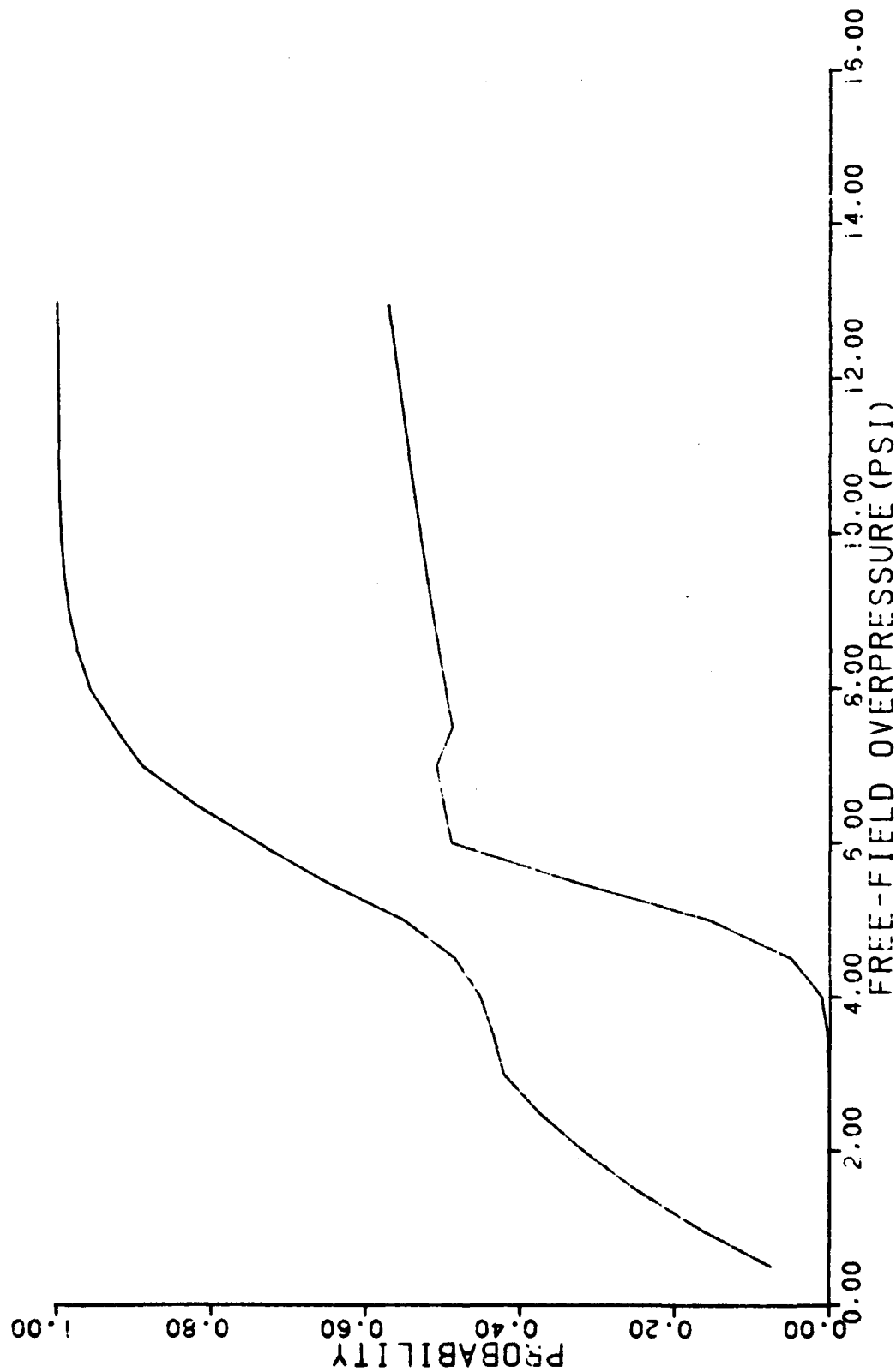


Figure C-23. Probability of slab failure (upper and lower bounds) case 3B.

CASE 3B3

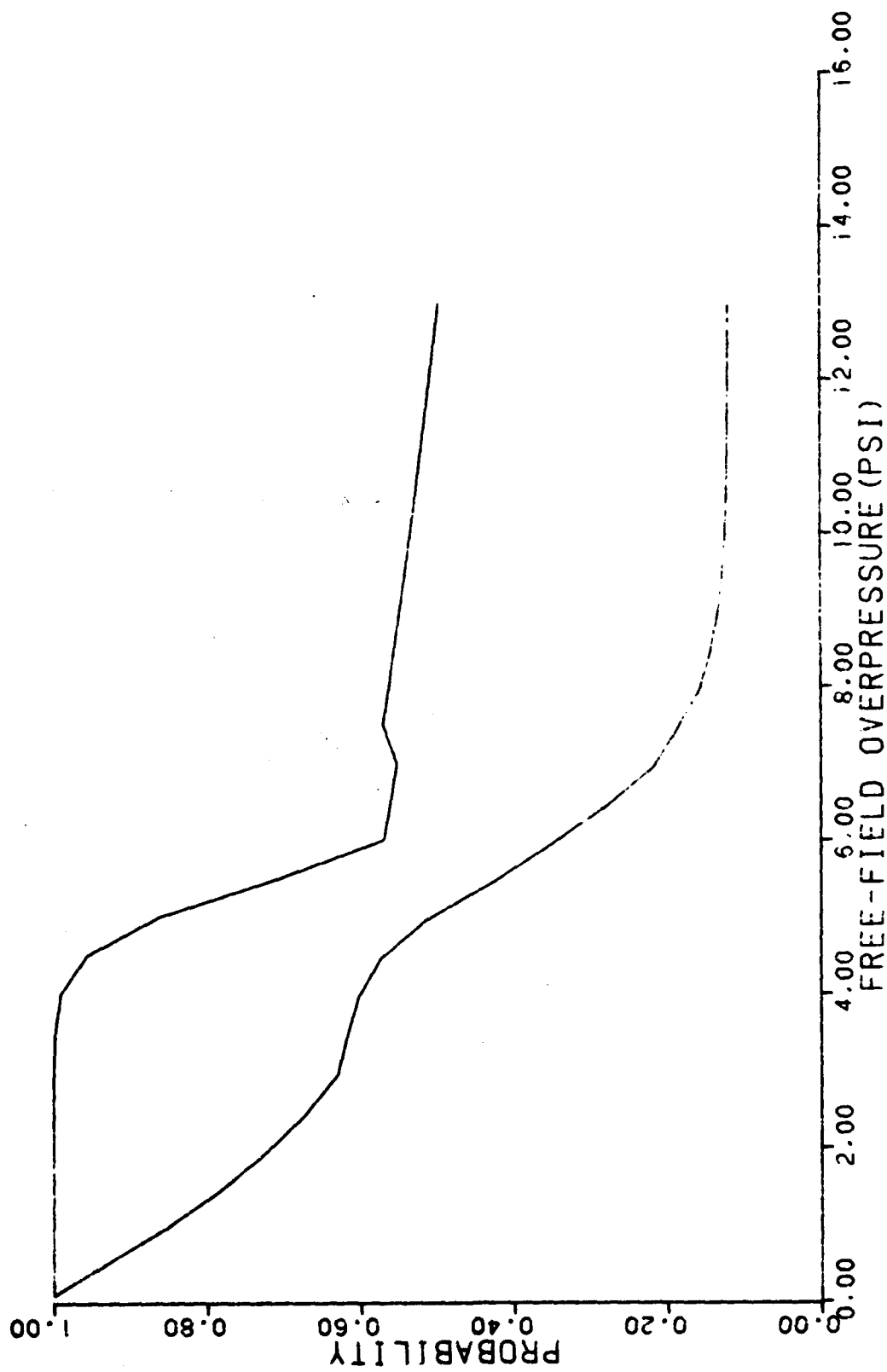


Figure C-24. Probability of people survival (upper and lower bounds) case 3B.

CASE 3C2

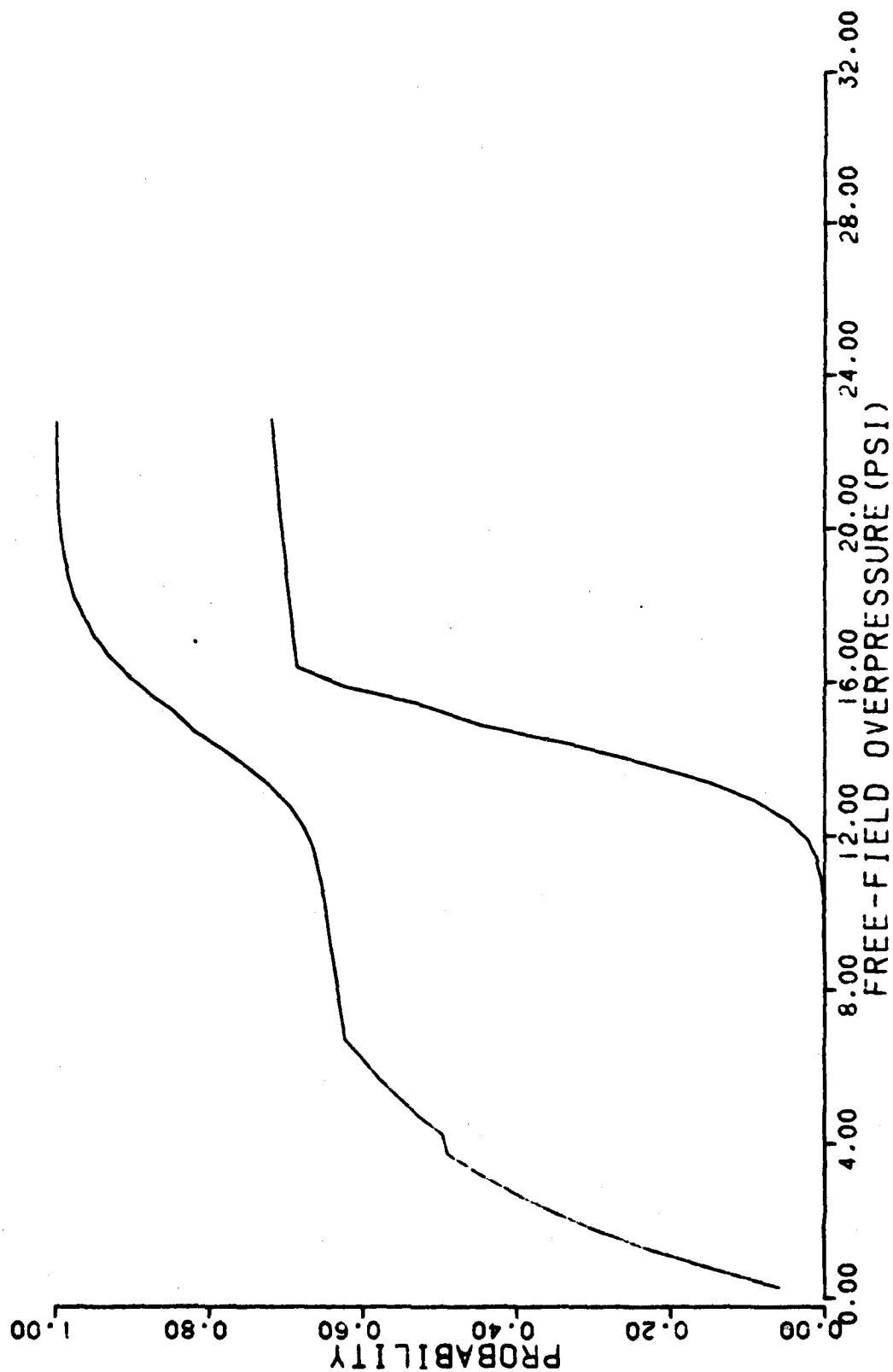


Figure C-25. Probability of slab failure (upper and lower bounds) case 3C.

CASE 3C3

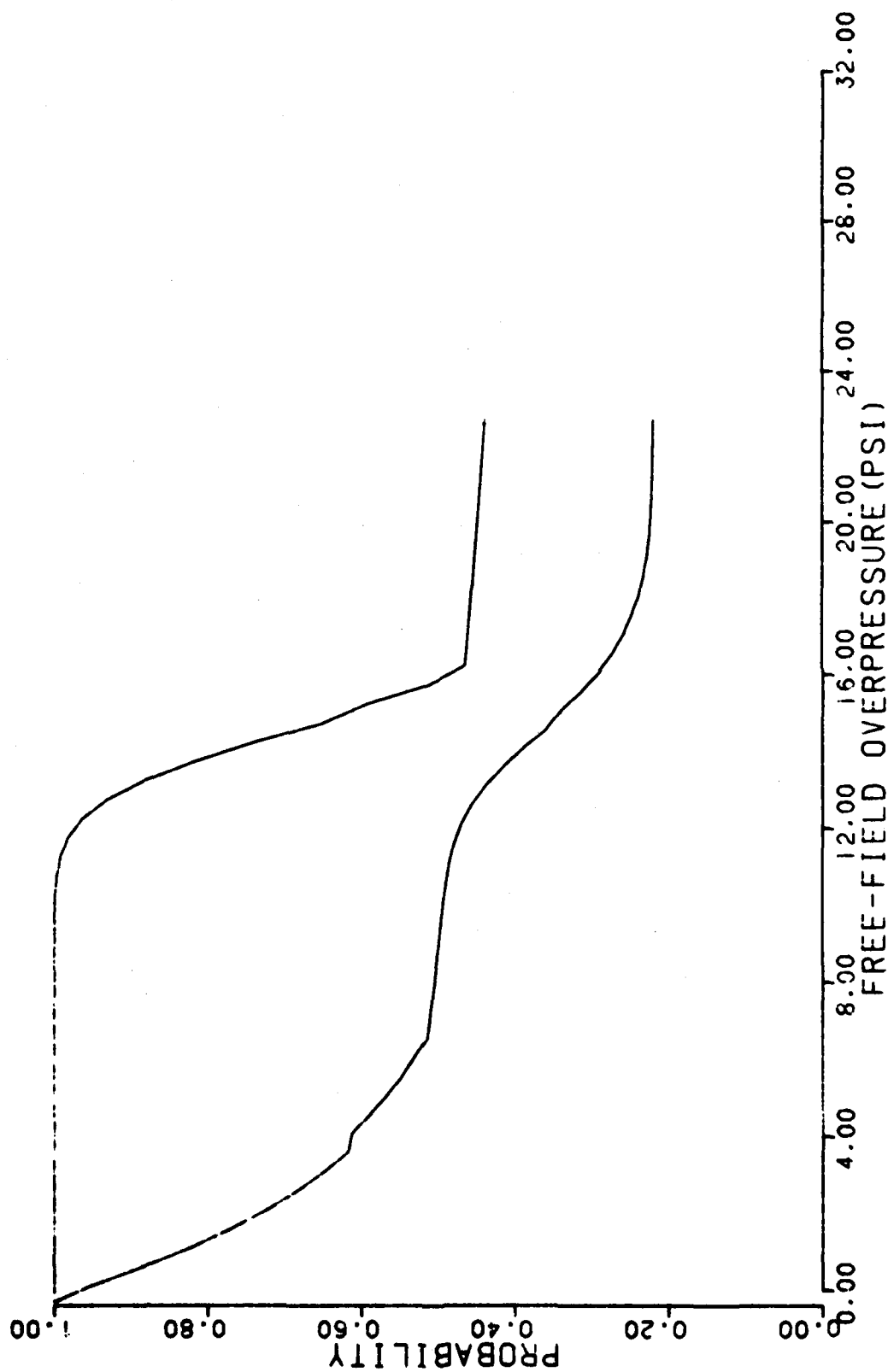


Figure C-26. Probability of people survival (upper and lower bounds) case 3C.

CASE 3D2

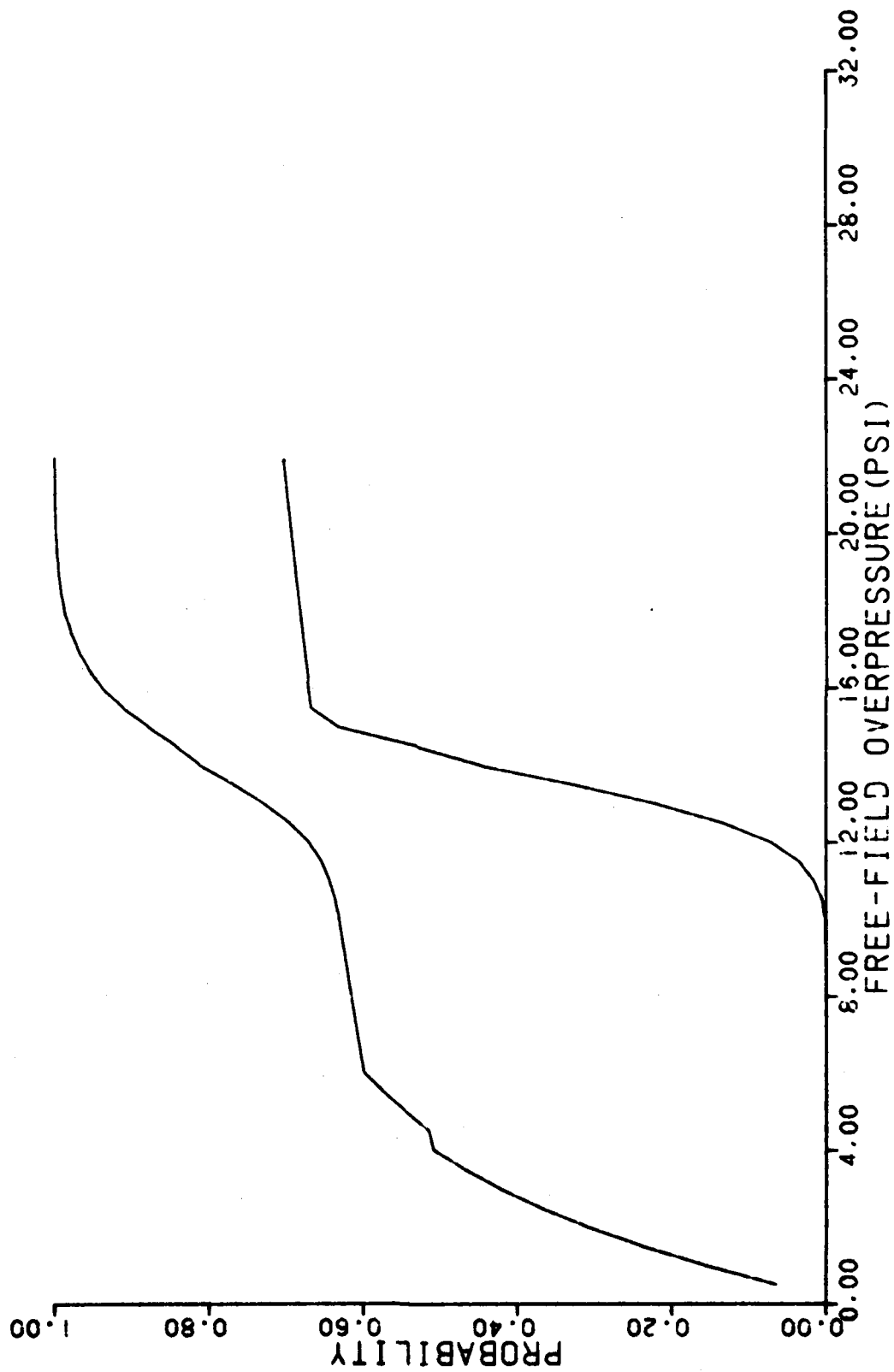


Figure C-27. Probability of slab failure (upper and lower bounds) case 3D.

CASE 3D3

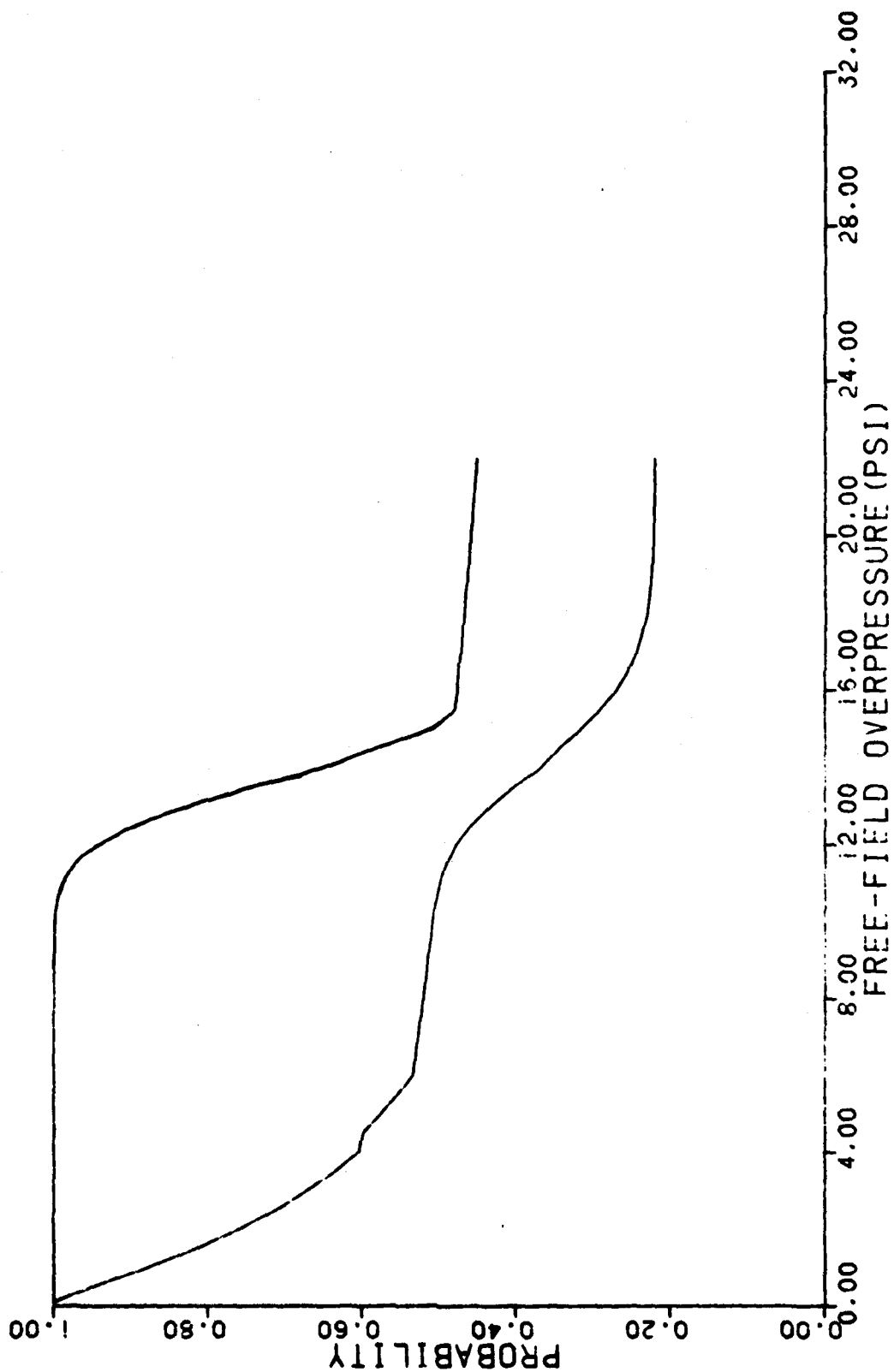


Figure C-28. Probability of people survival (upper and lower bounds) case 3D.

CASE 3E2

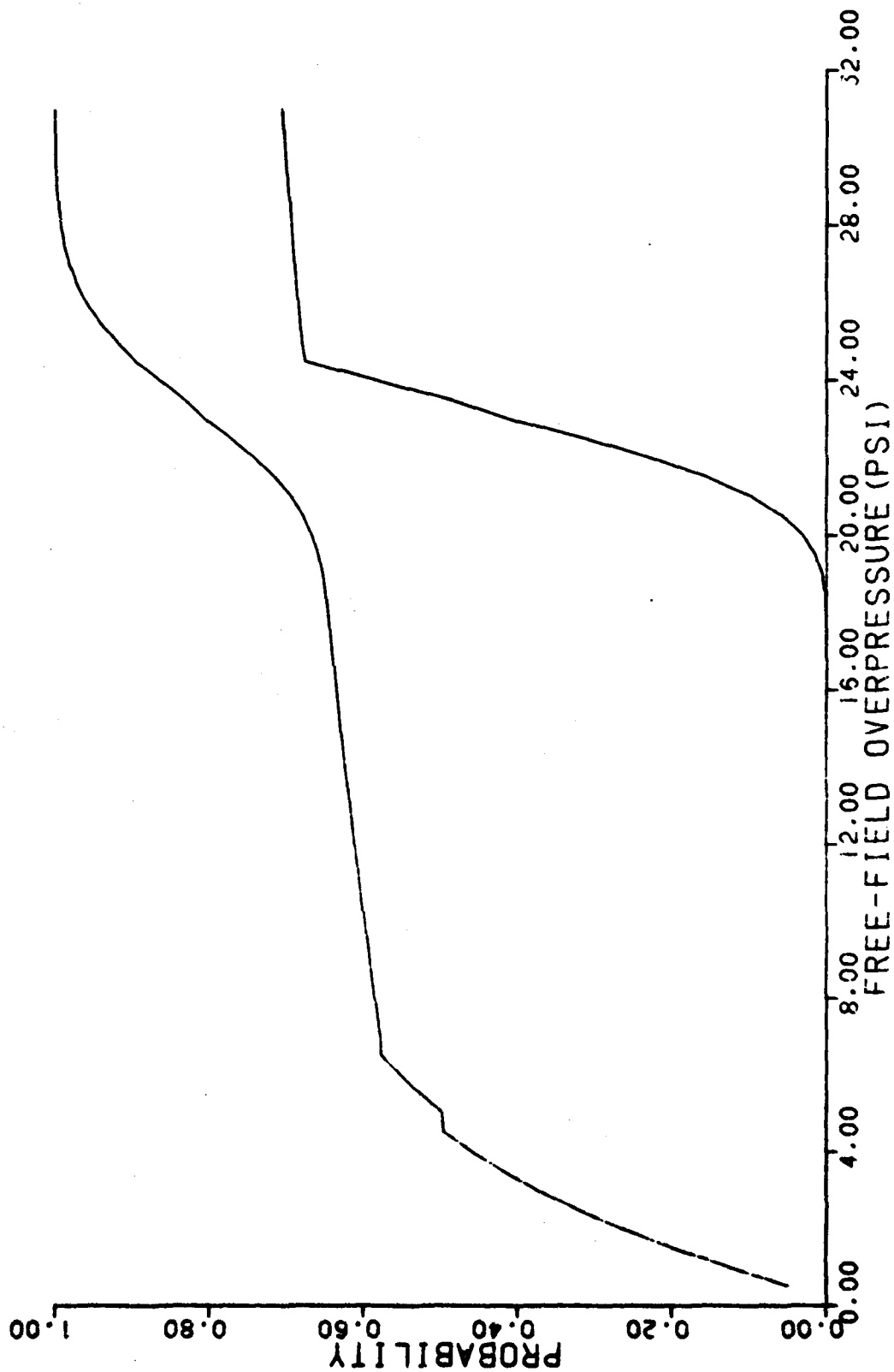


Figure C-29. Probability of slab failure (upper and lower bounds) case 3E.

CASE 3E3

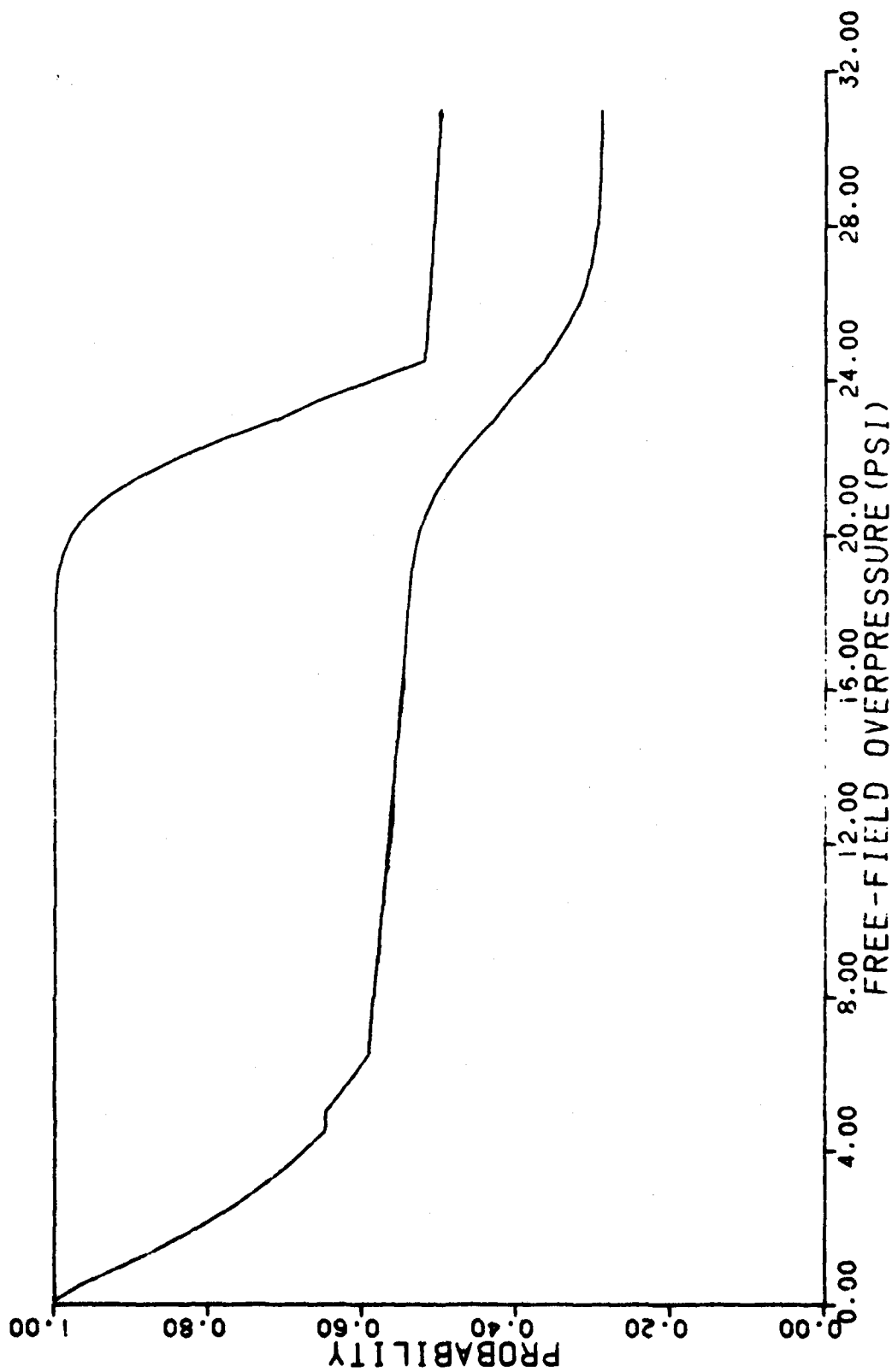


Figure C-30. Probability of people survival (upper and lower bounds) case 3E.

CASE 4A2

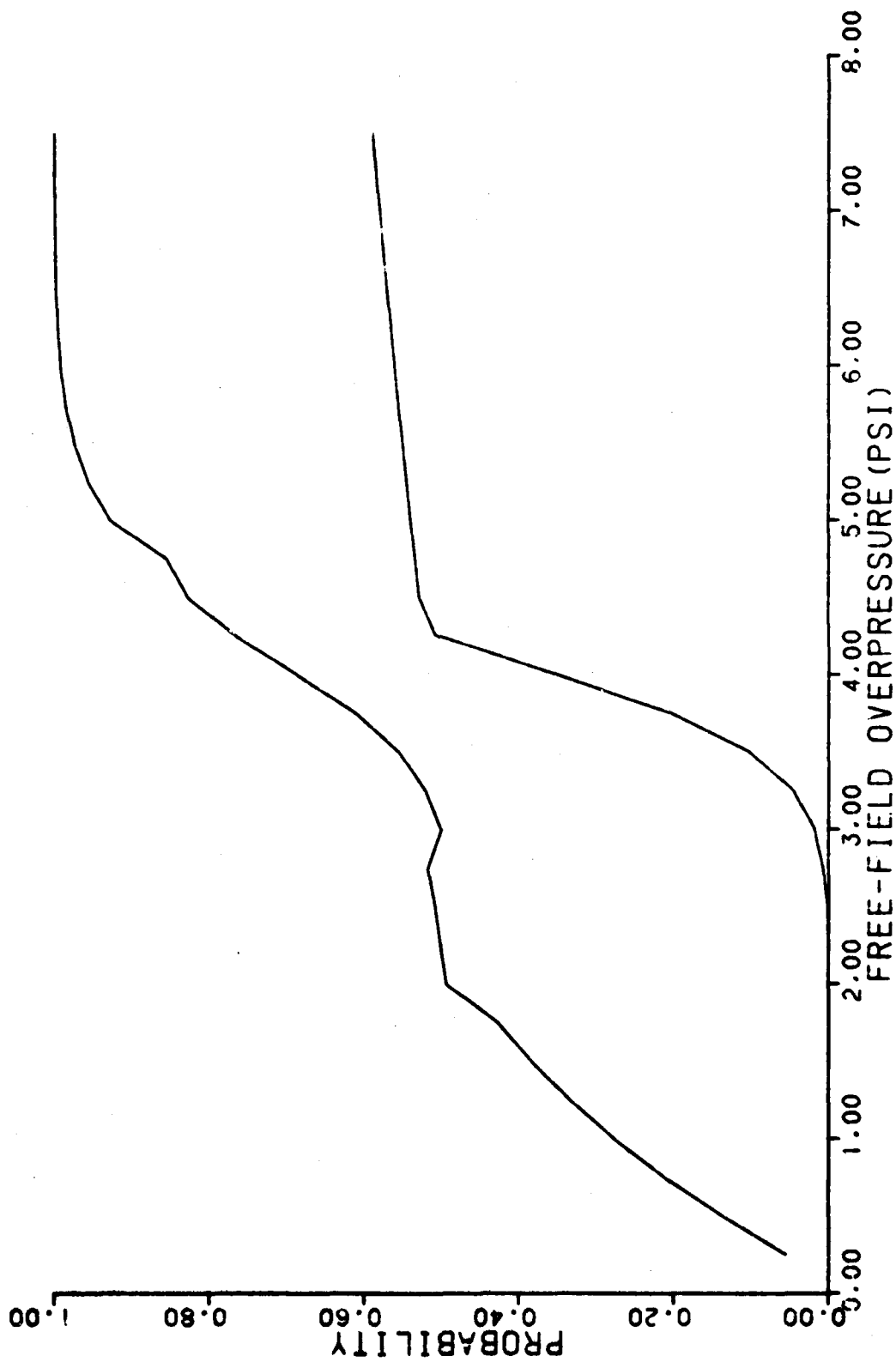


Figure C-31. Probability of slab failure (upper and lower bounds) case 4A.

CASE 4A3

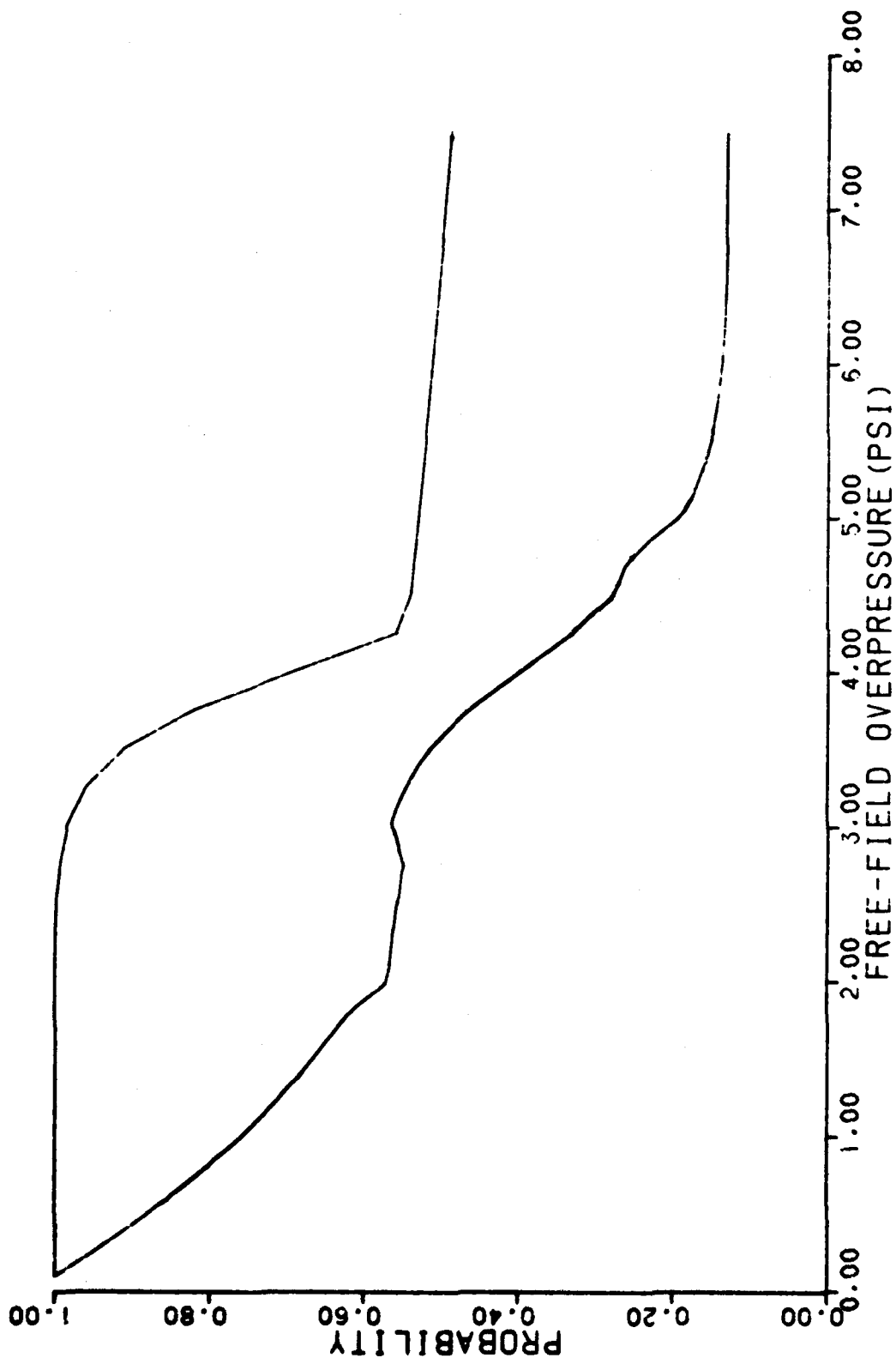


Figure C-32. Probability of people survival (upper and lower bounds) case 4A.

CASE 4B2

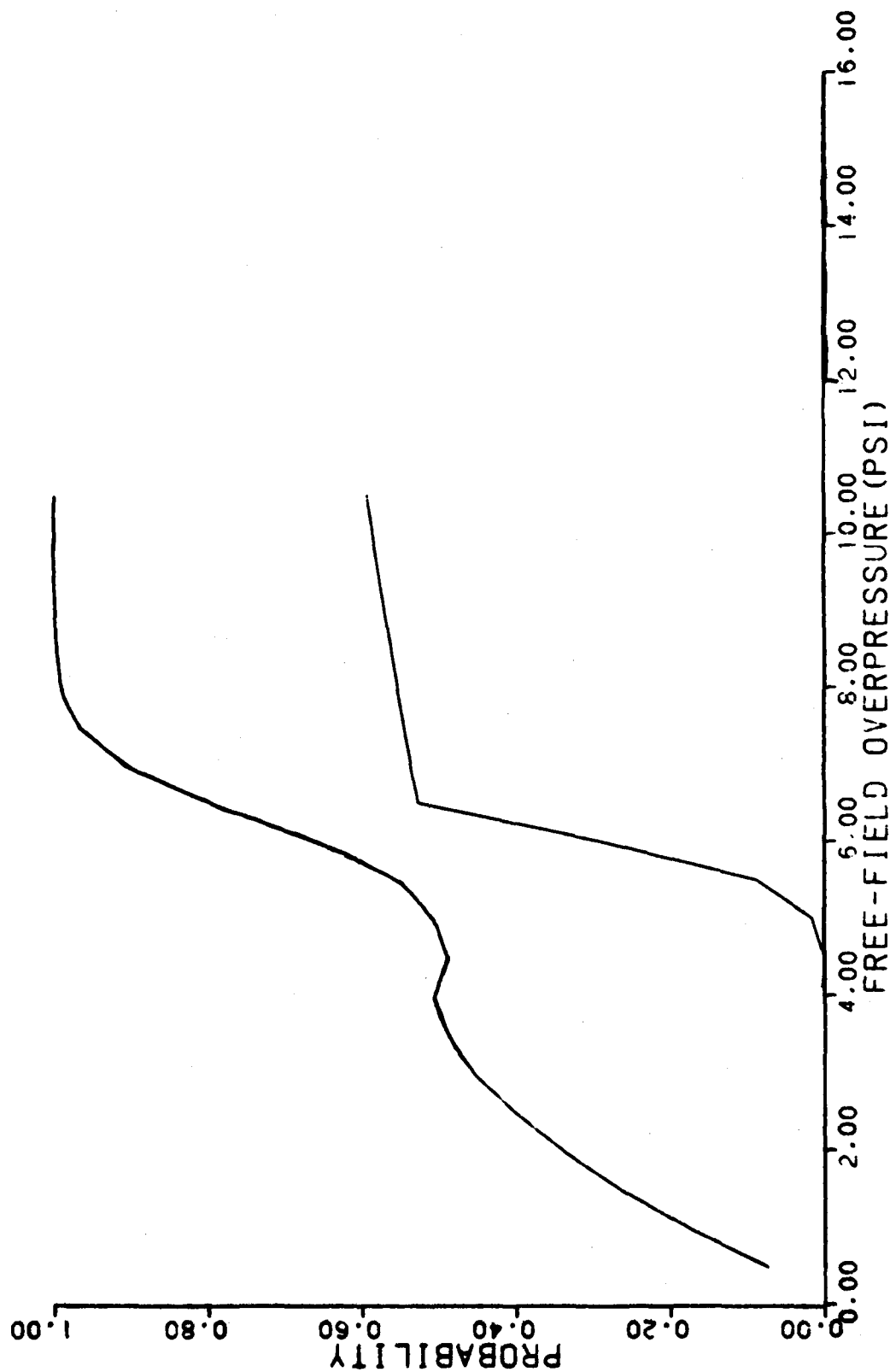


Figure C-33. Probability of slab failure (upper and lower bounds) case 4B.

CASE 4B3

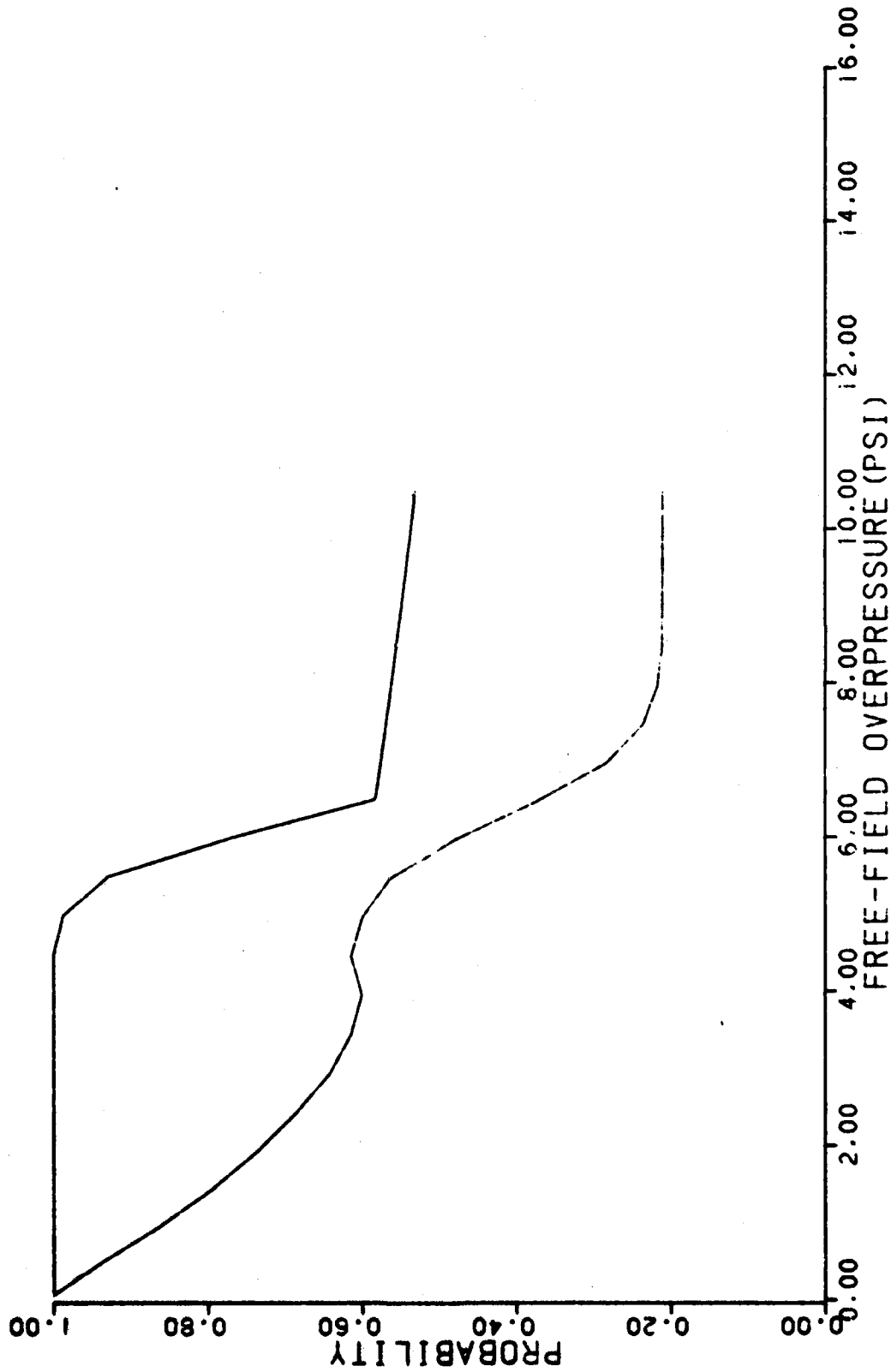


Figure C-34. Probability of people survival (upper and lower bounds) case 4B.

CASE 4C2

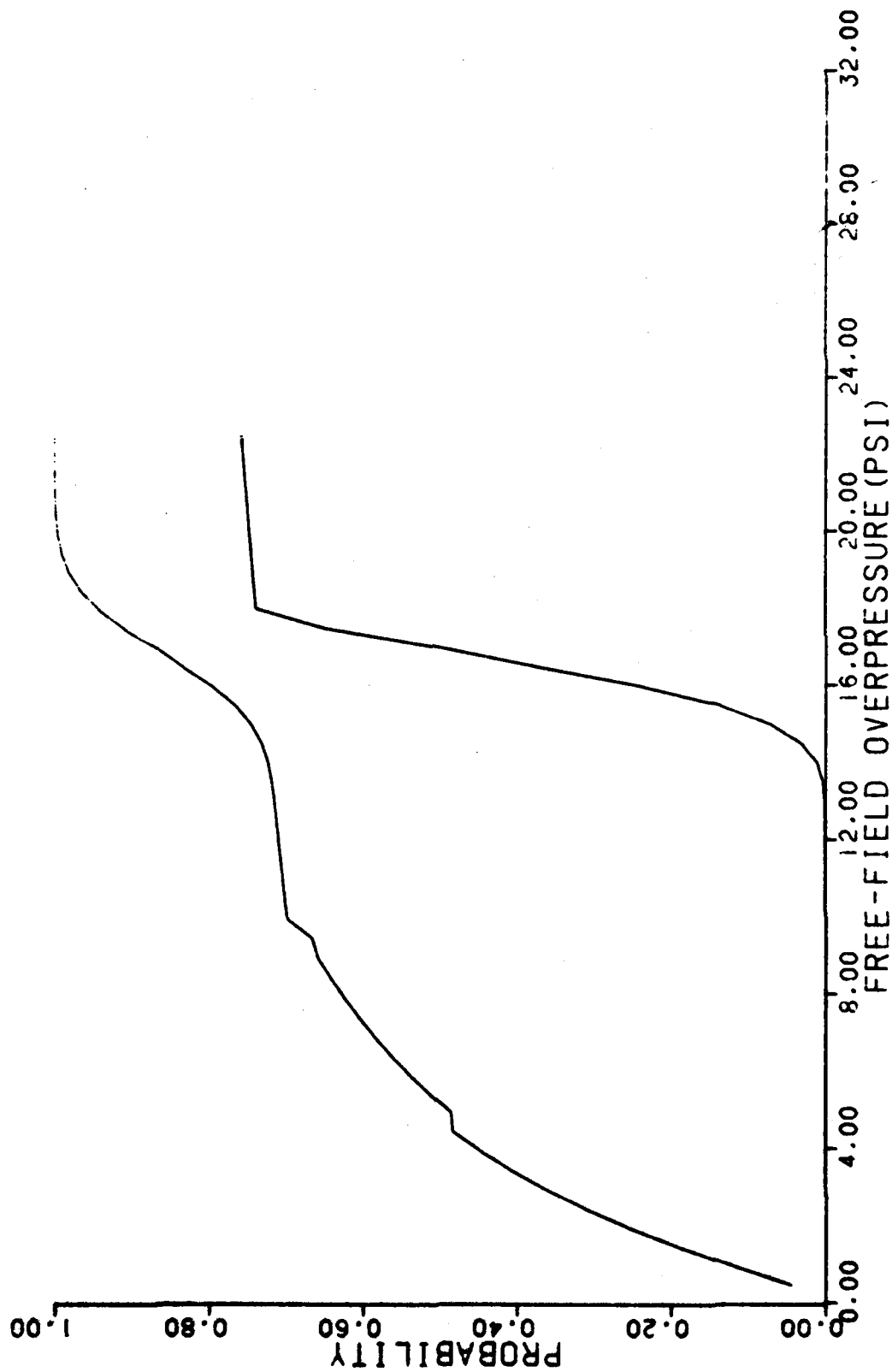


Figure C-35. Probability of slab failure (upper and lower bounds) case 4C.

CASE 4C3

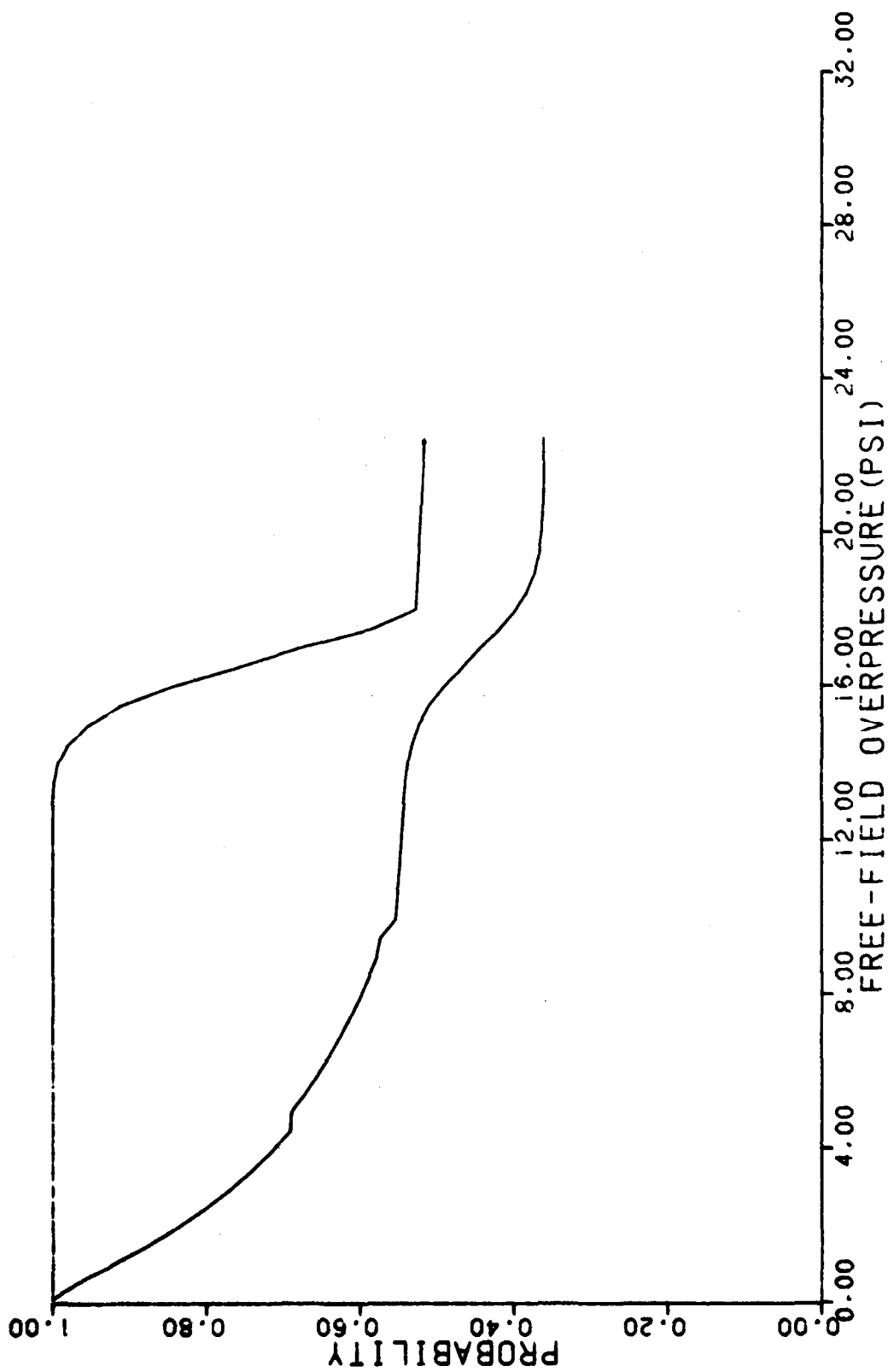


Figure C-36. Probability of people survival (upper and lower bounds) case 4C.

CASE 4D2

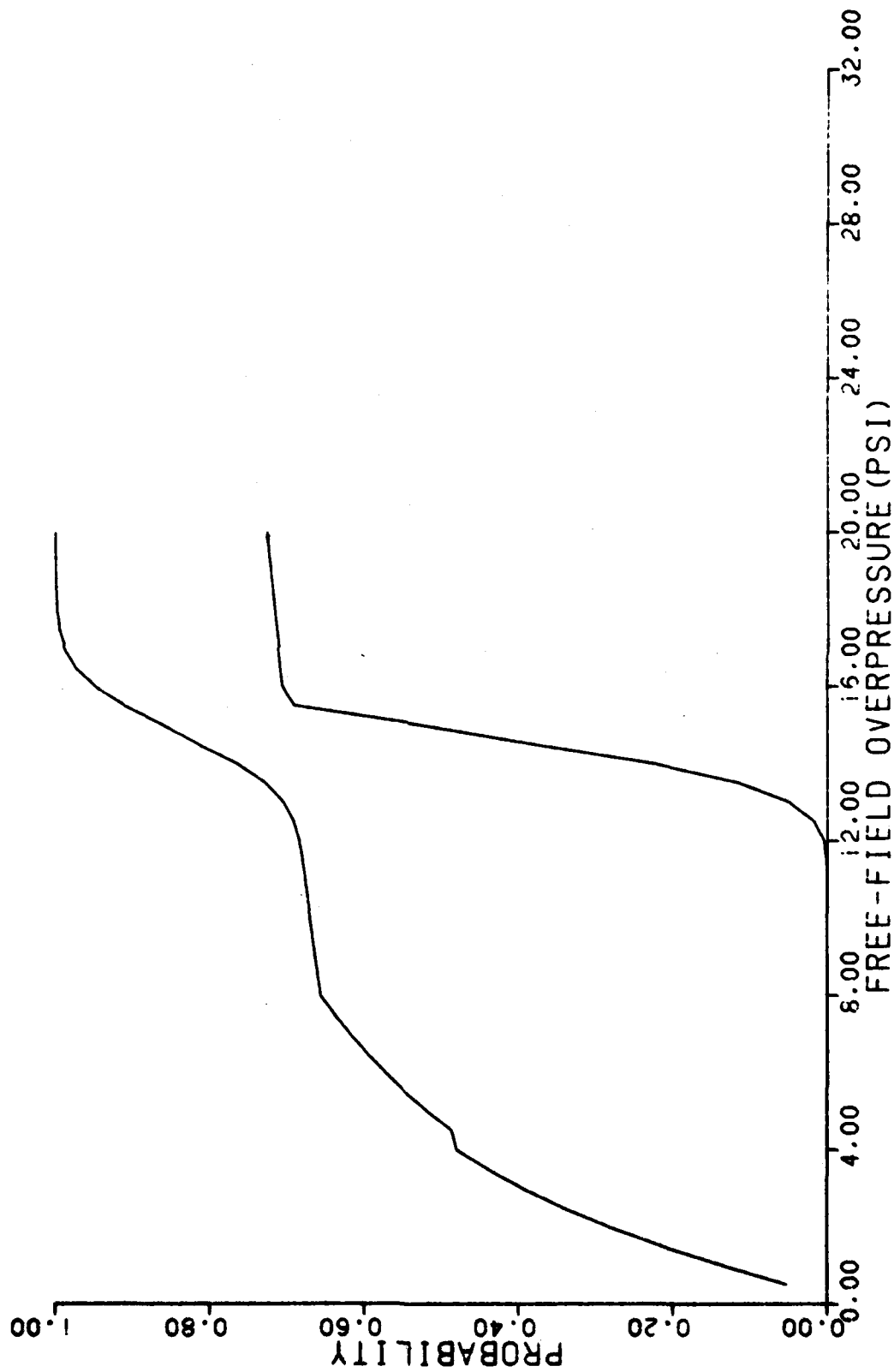


Figure C-37. Probability of slab failure (upper and lower bounds) case 4D.

CASE 4D3

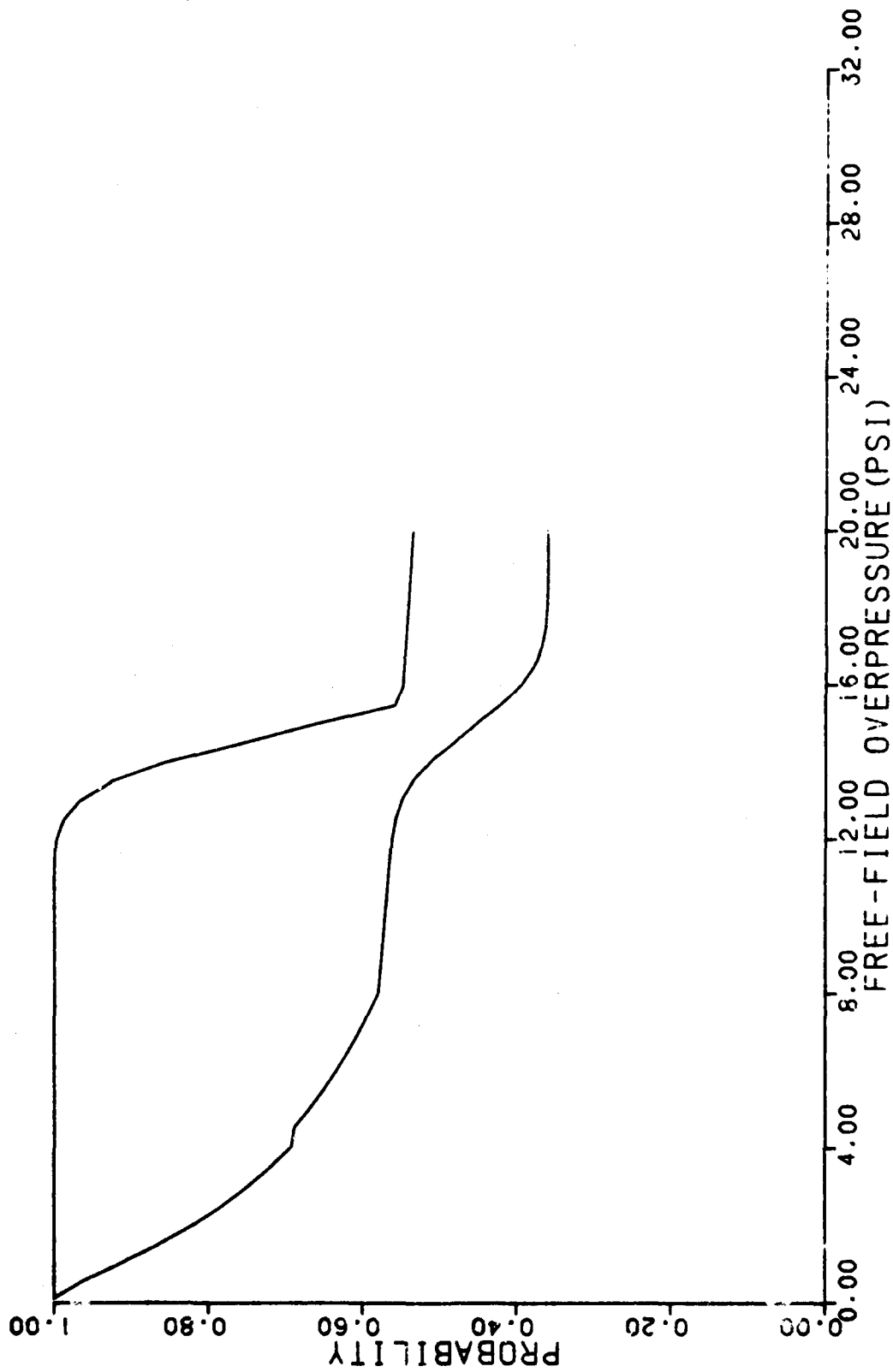


Figure C-38. Probability of people survival (upper and lower bounds) case 4D.

CASE 4E2

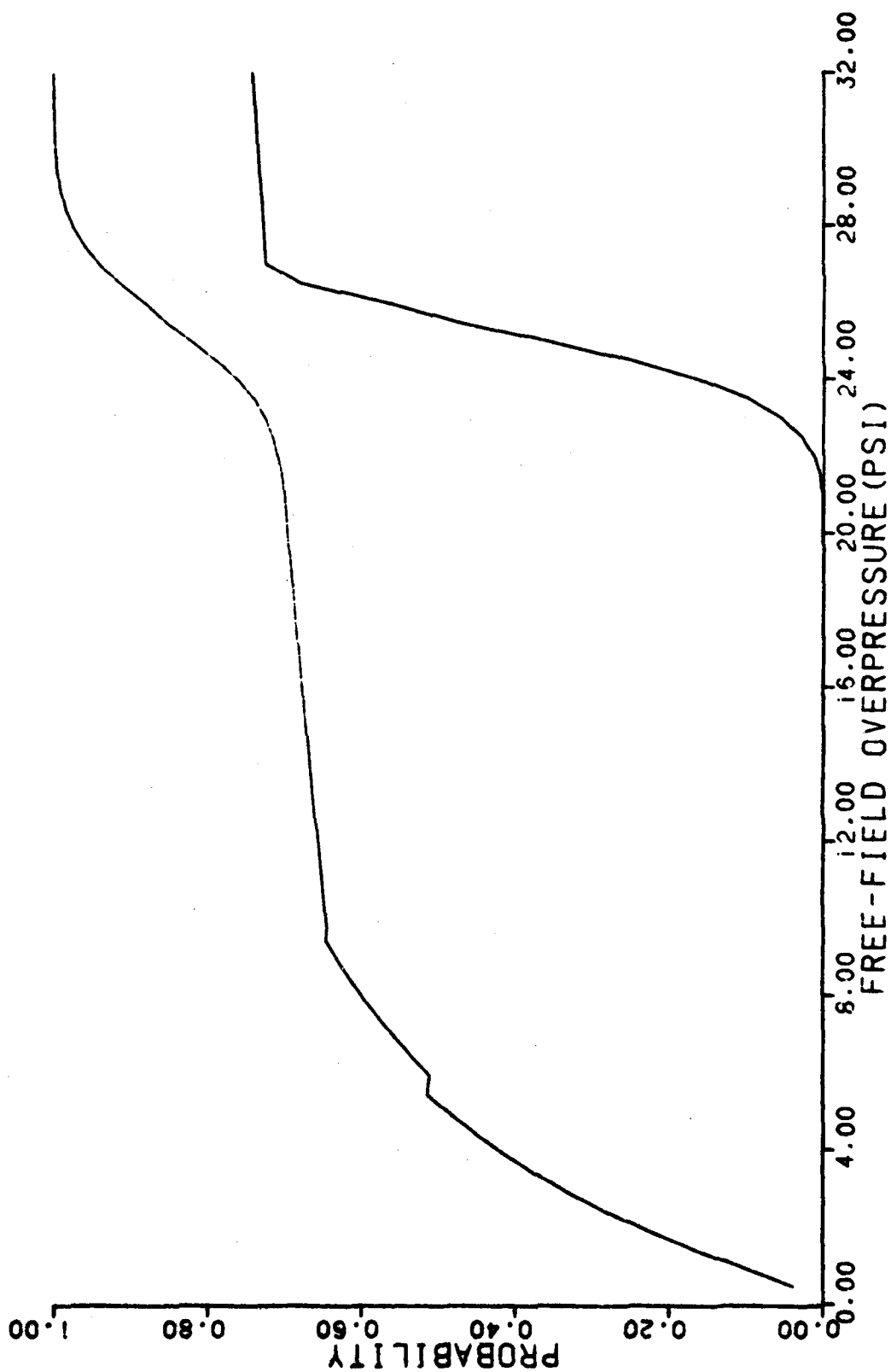


Figure C-39. Probability of slab failure (upper and lower bounds) case 4E.

CASE 4E3

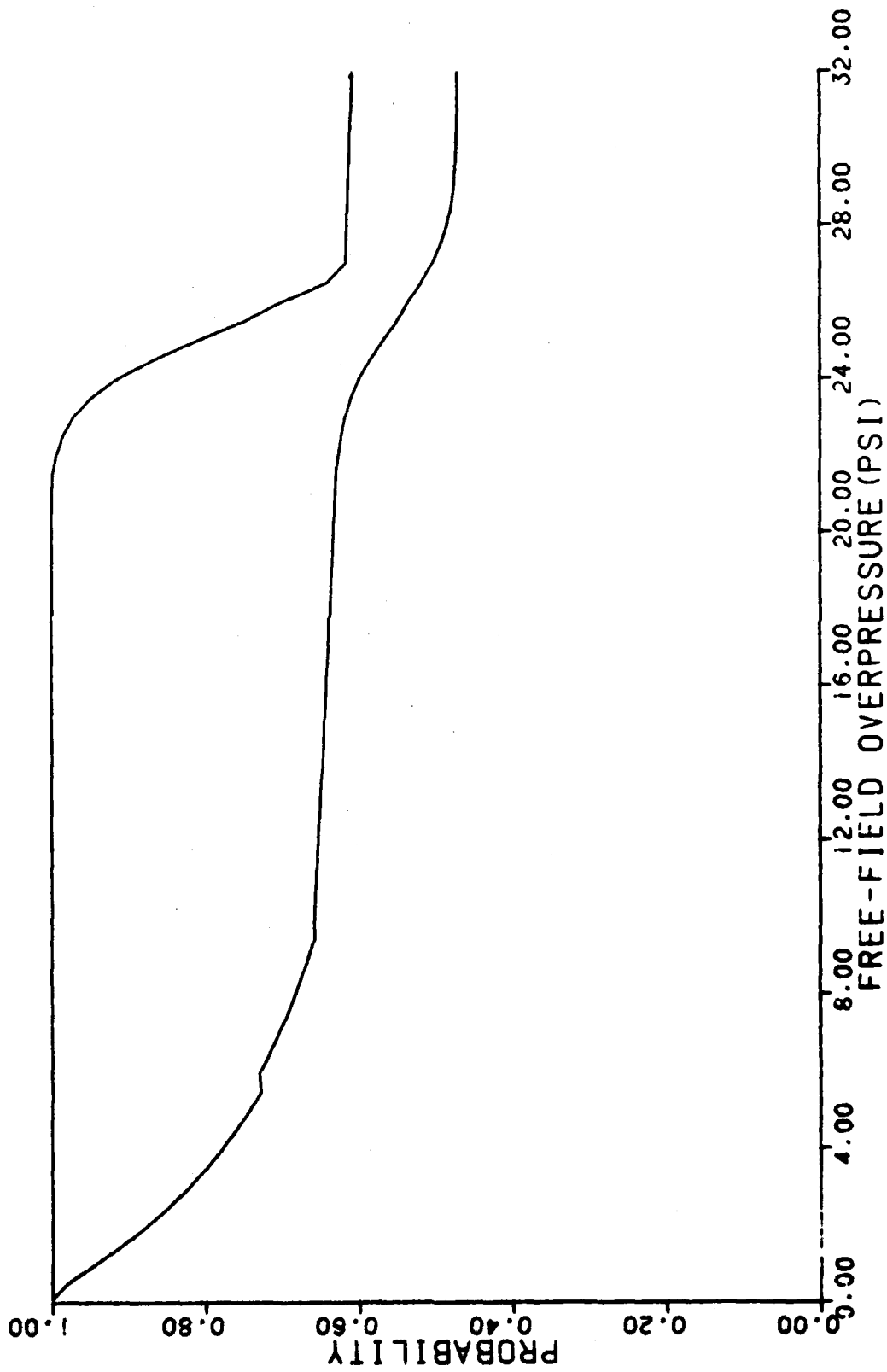


Figure C-40. Probability of people survival (upper and lower bounds) case 4E.

CASE 5A2

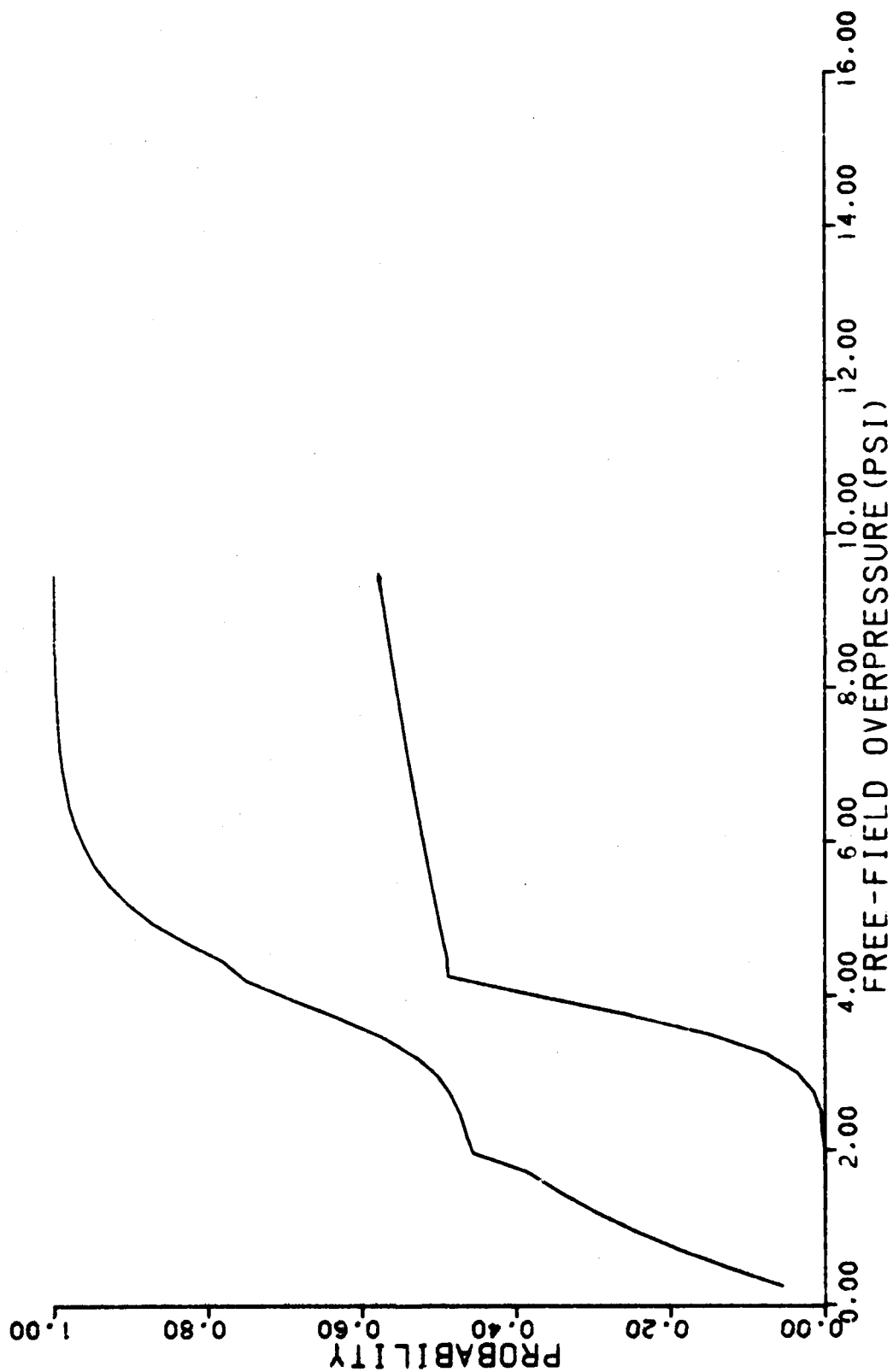


Figure C-41. Probability of slab failure (upper and lower bounds) case 5A.

CASE 5A3

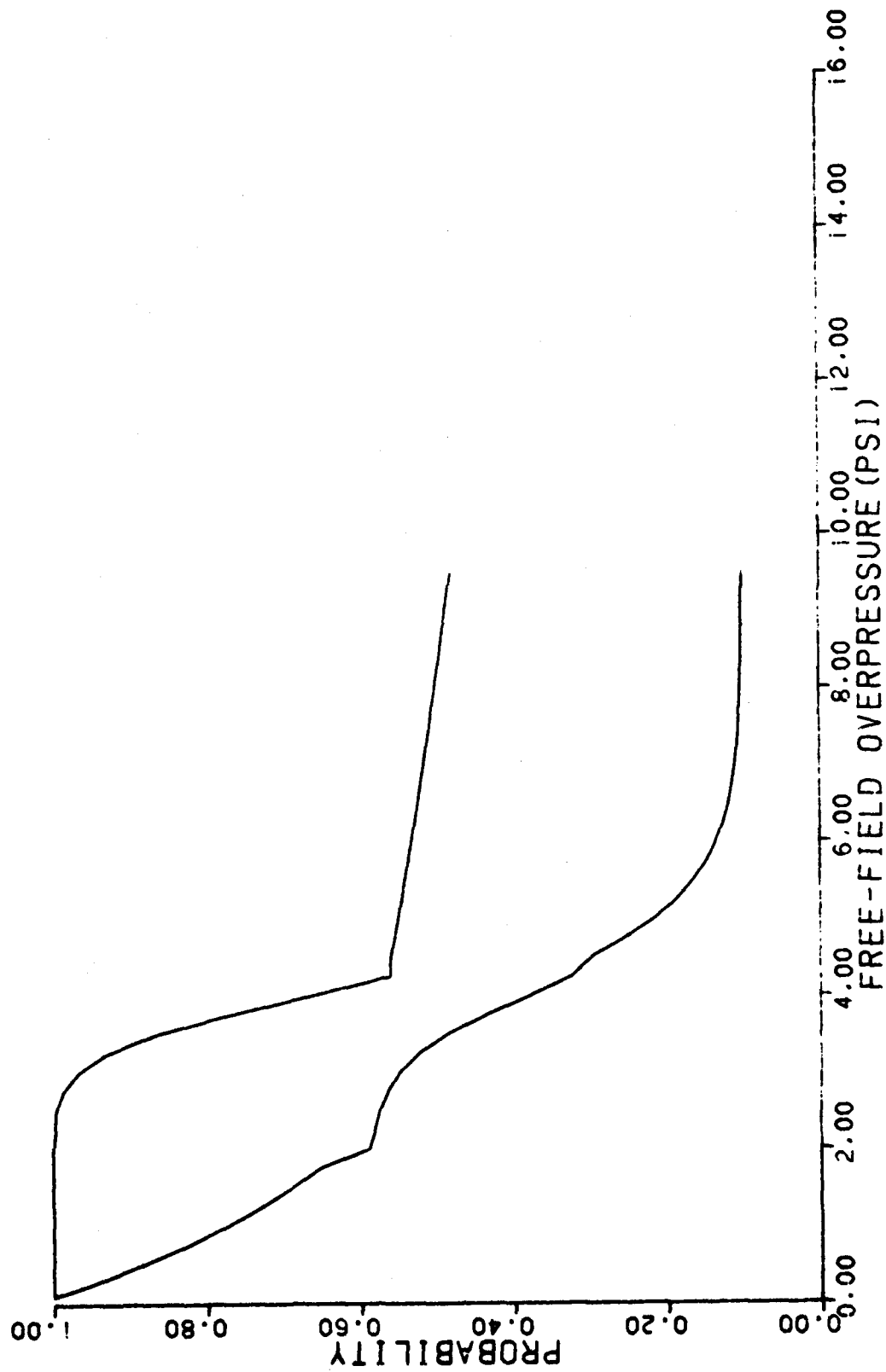
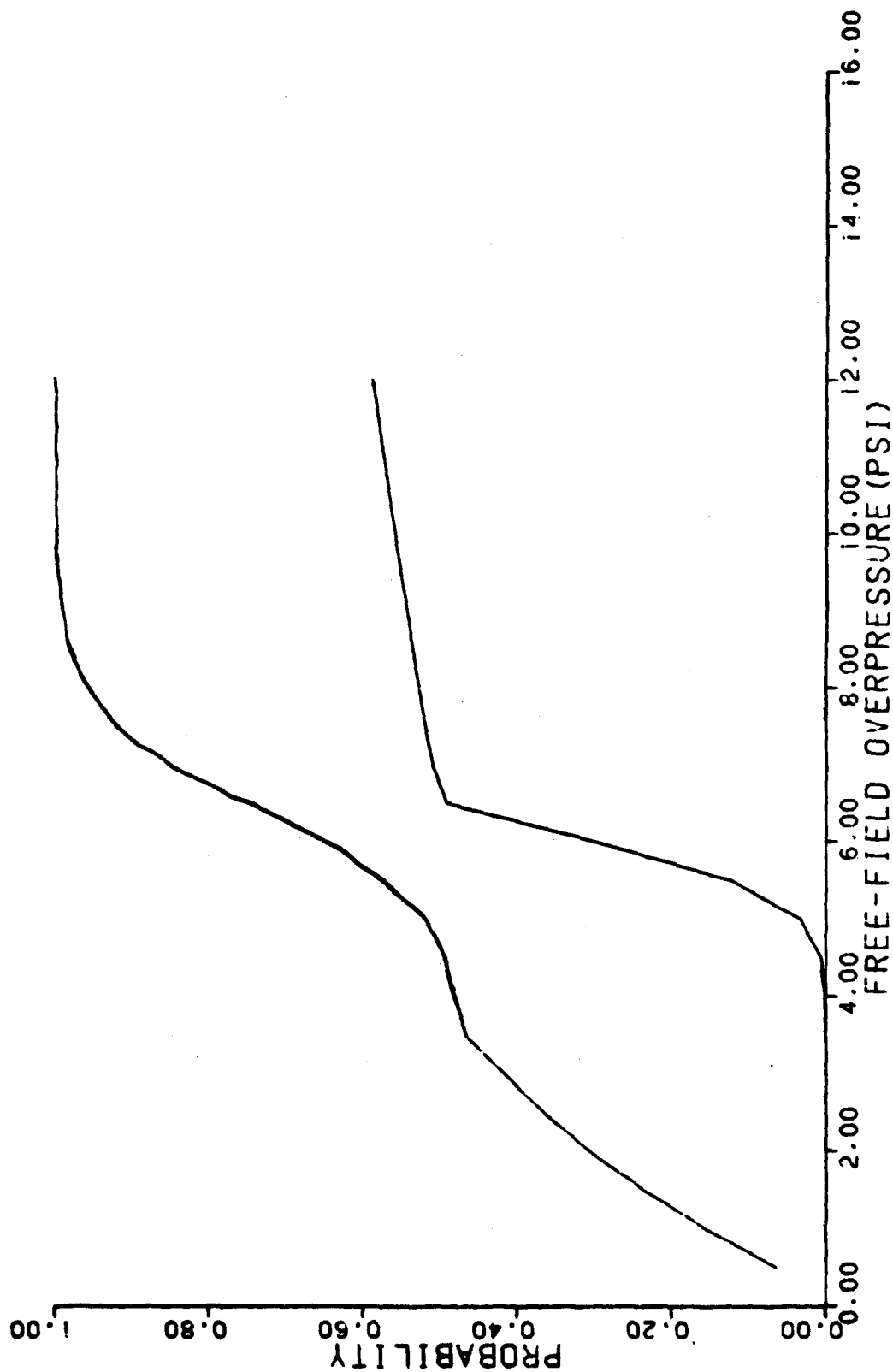


Figure C-42. Probability of people survival (upper and lower bounds) case 5A.

CASE 5B2



C-43. Probability of slab failure (upper and lower bounds) case 5B.

CASE 5B3

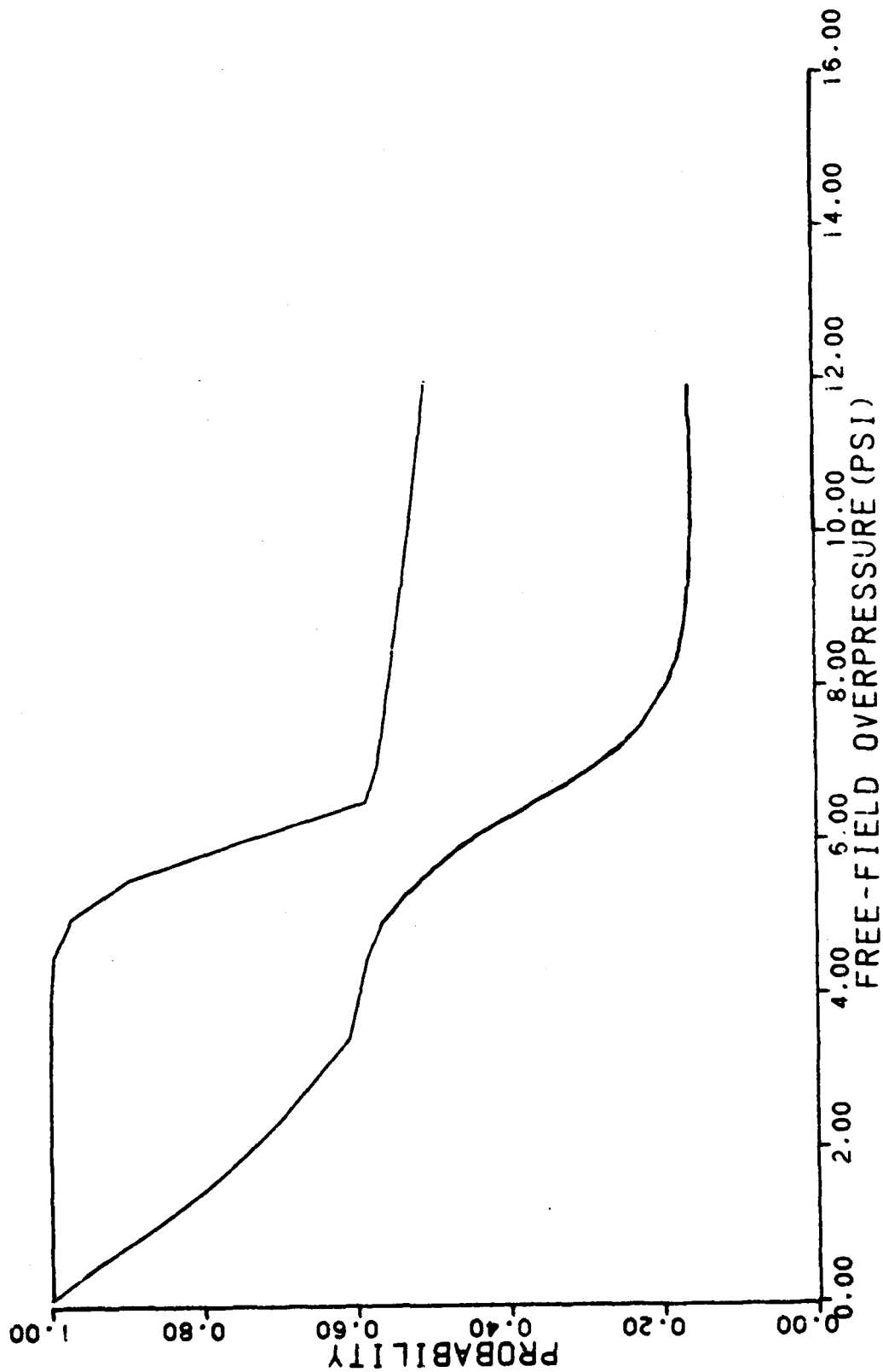


Figure C-44. Probability of people survival (upper and lower bounds) case 5B.

CASE 5C2



Figure C-45. Probability of slab failure (upper and lower bounds) case 5C.

CASE 5C3

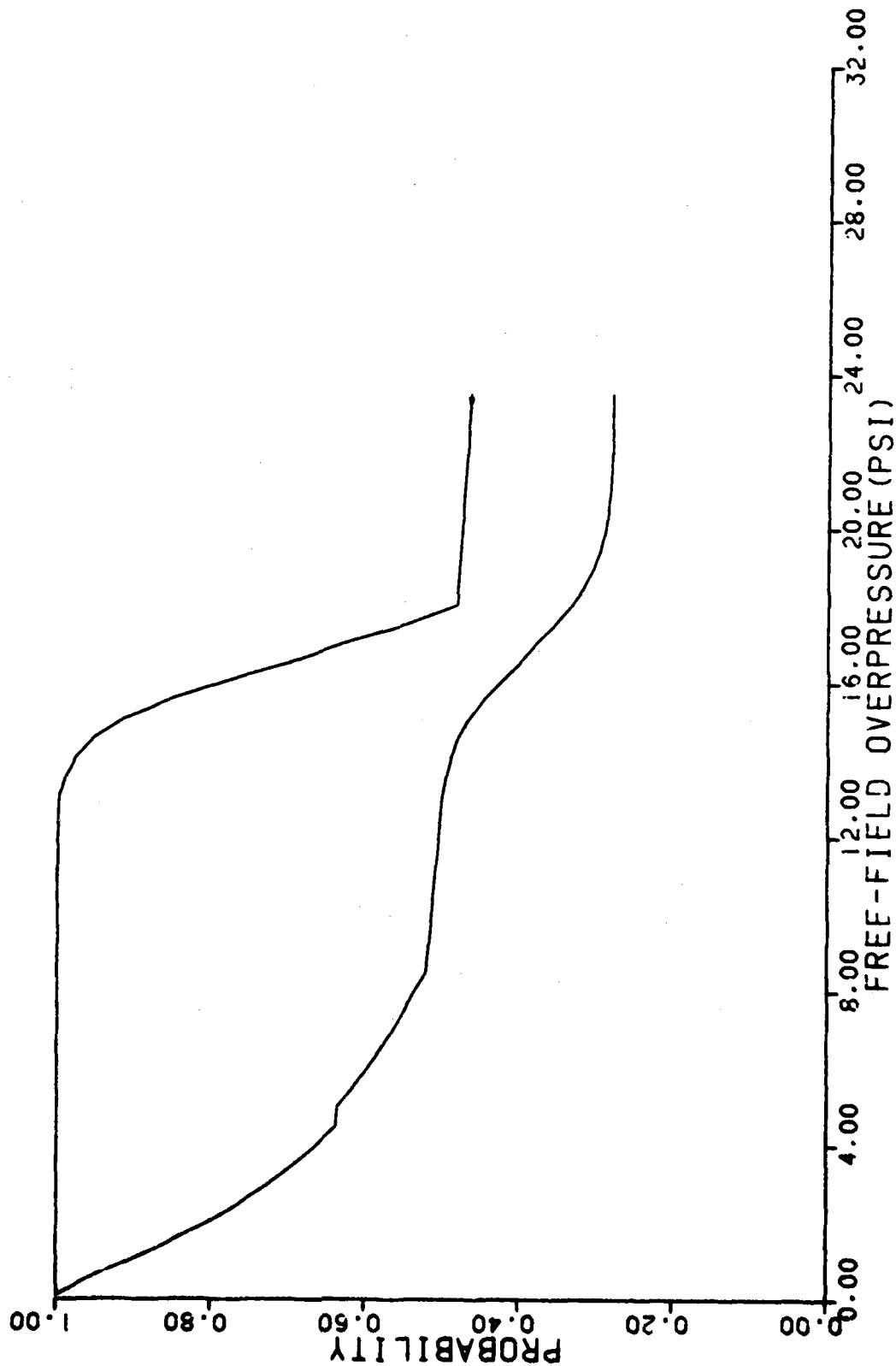


Figure C-46. Probability of people survival (upper and lower bounds) case 5C.

CASE 5D2

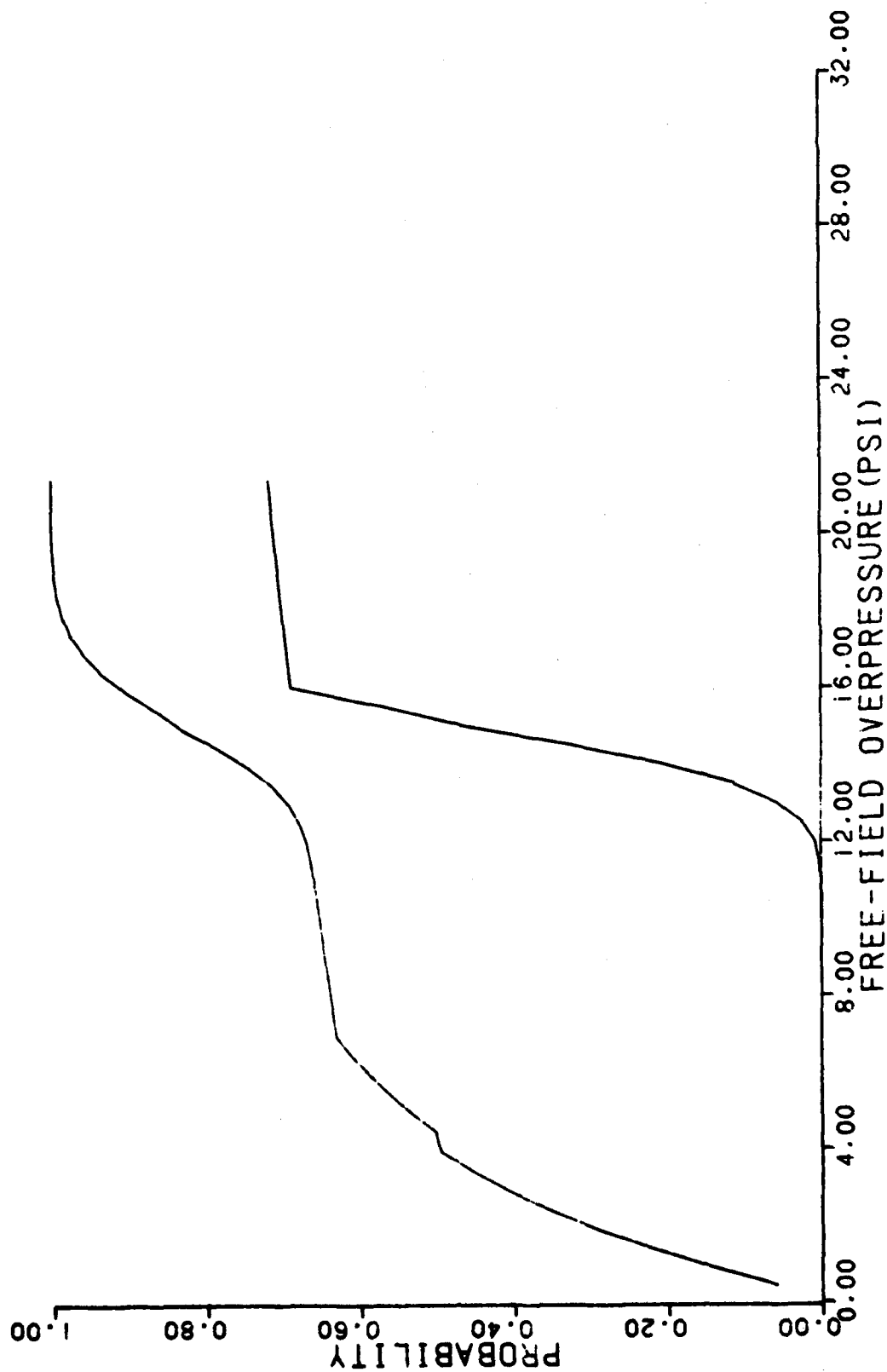


Figure C-47. Probability of slab failure (upper and lower bounds) case 5D.

CASE 5D3

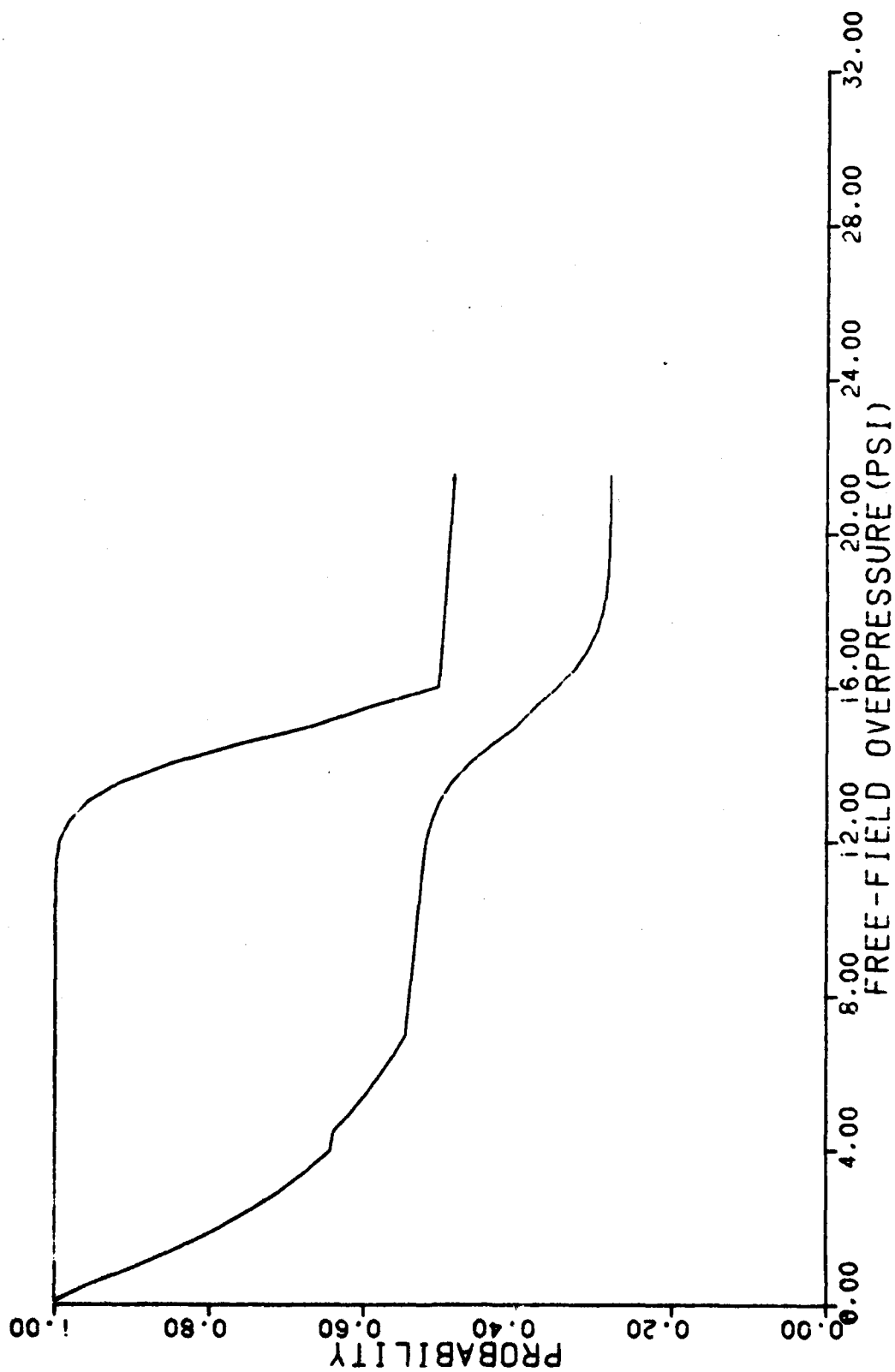


Figure C-48. Probability of people survival (upper and lower bounds) case 5D.

CASE 5E2

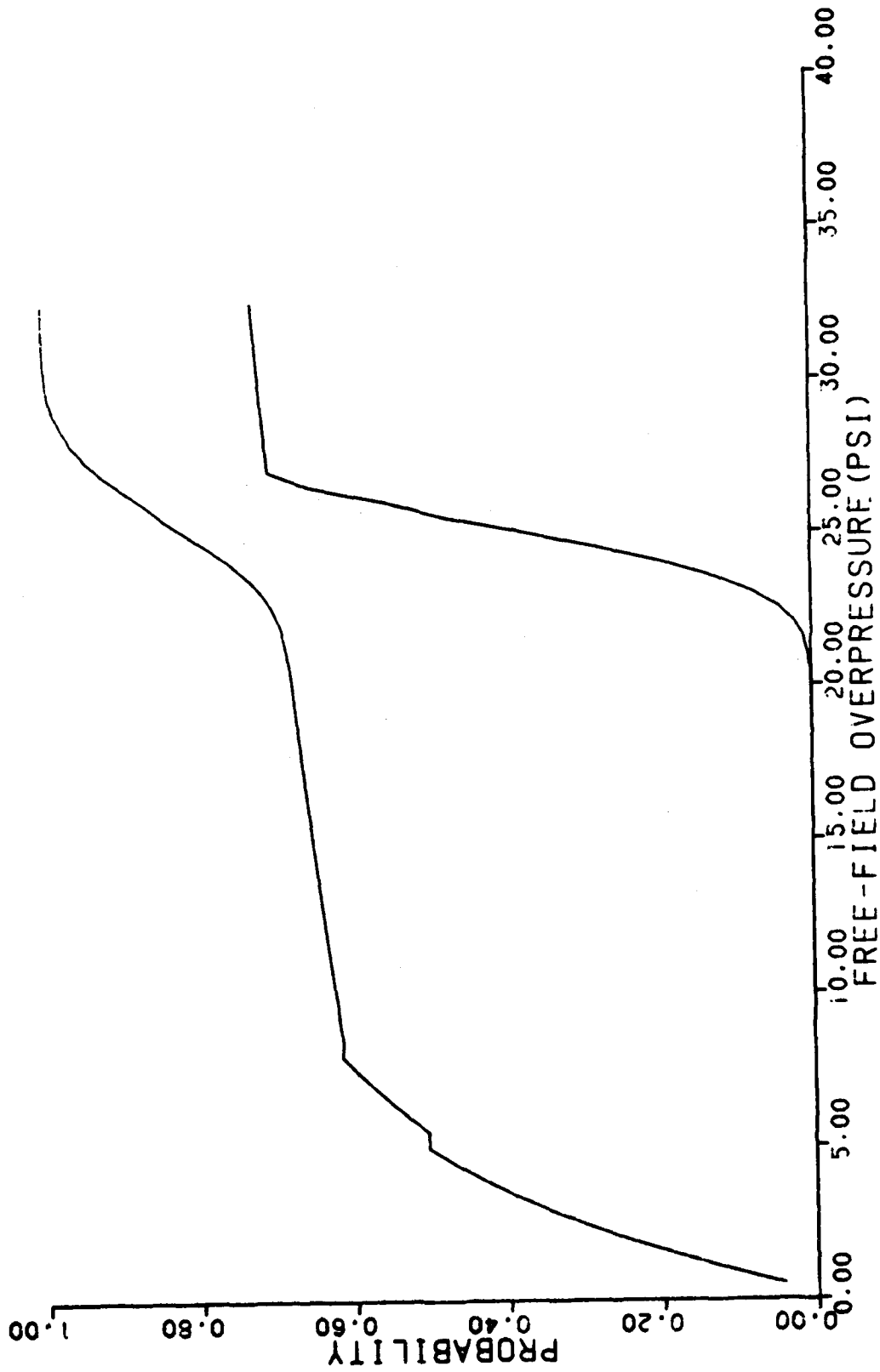


Figure C-49. Probability of slab failure (upper and lower bounds) case 5E.

CASE 5E3

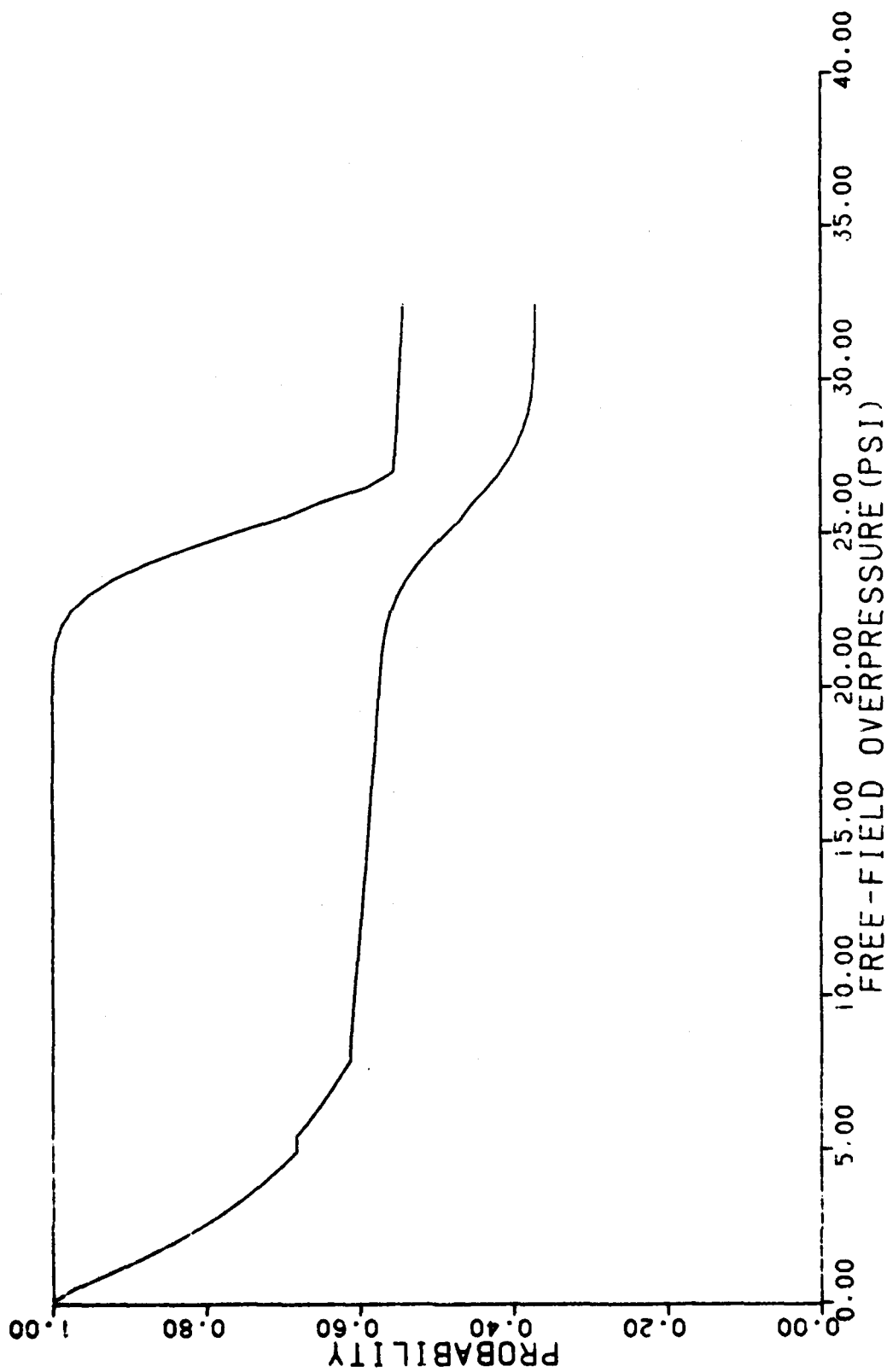


Figure C-50. Probability of people survival (upper and lower bounds) case 5E.

CASE 6A2

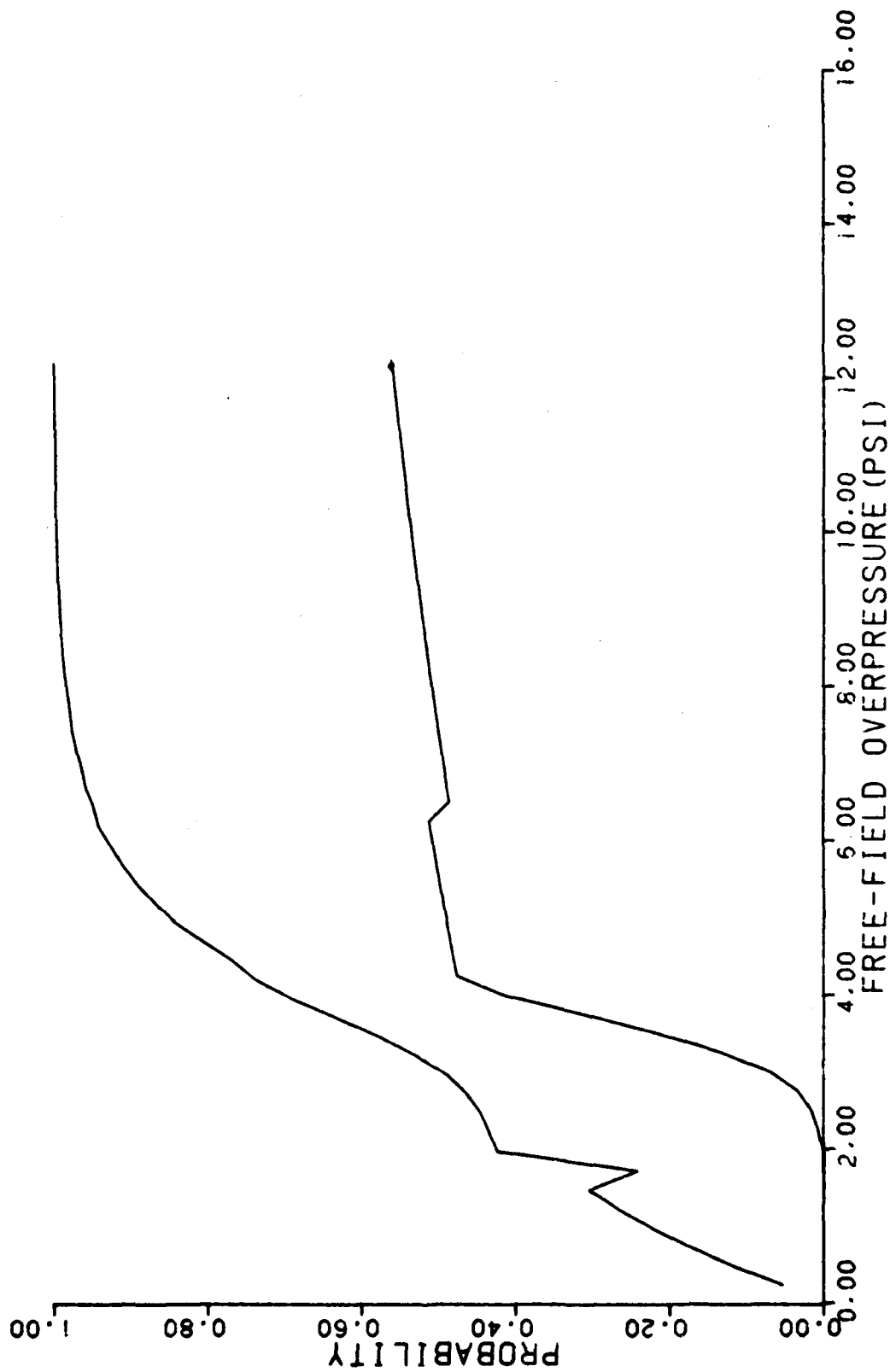


Figure C-51. Probability of slab failure (upper and lower bounds) case 6A.

CASE 6A3

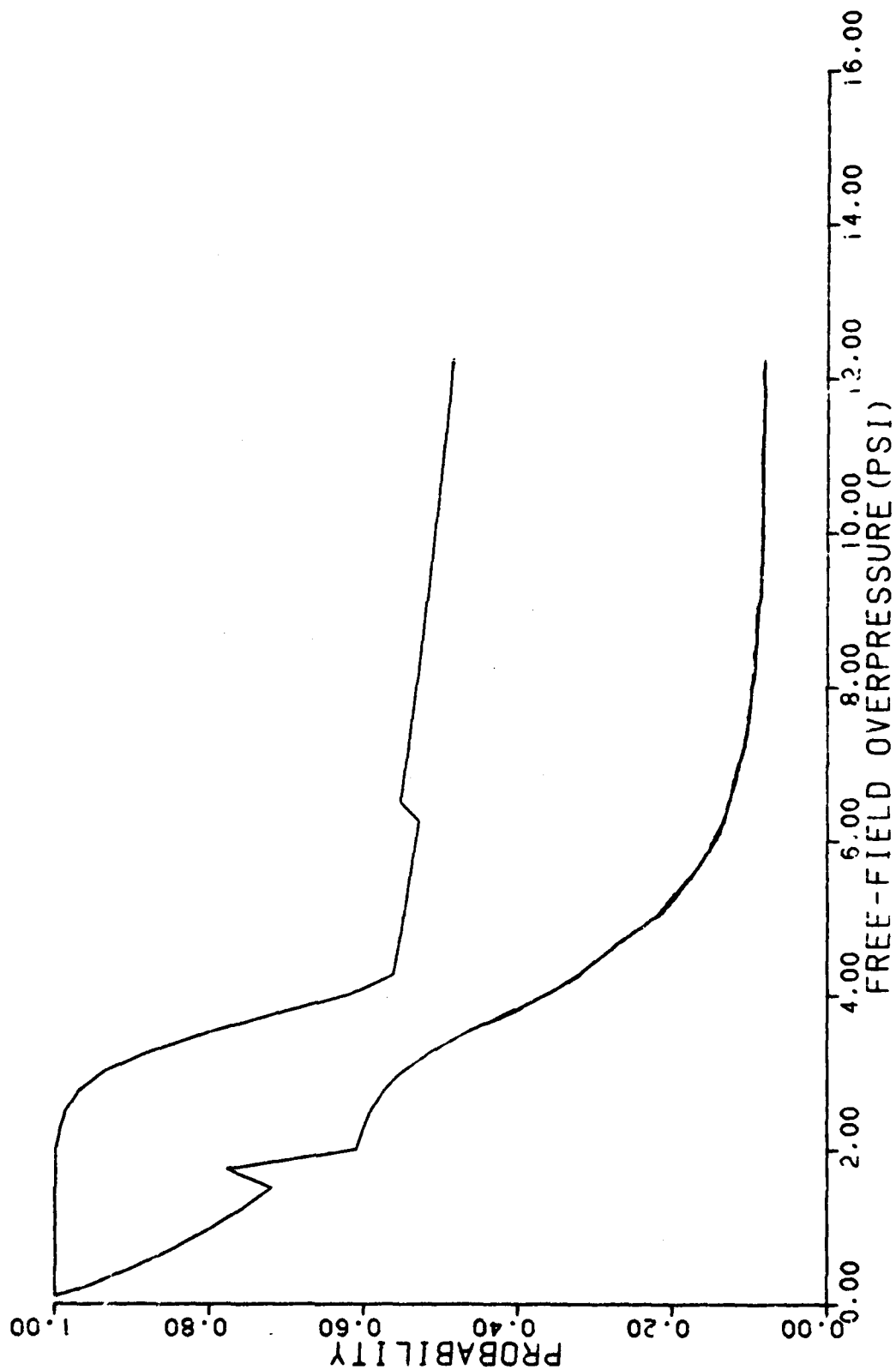


Figure C-52. Probability of people survival (upper and lower bounds) case 6A.

CASE 6B2

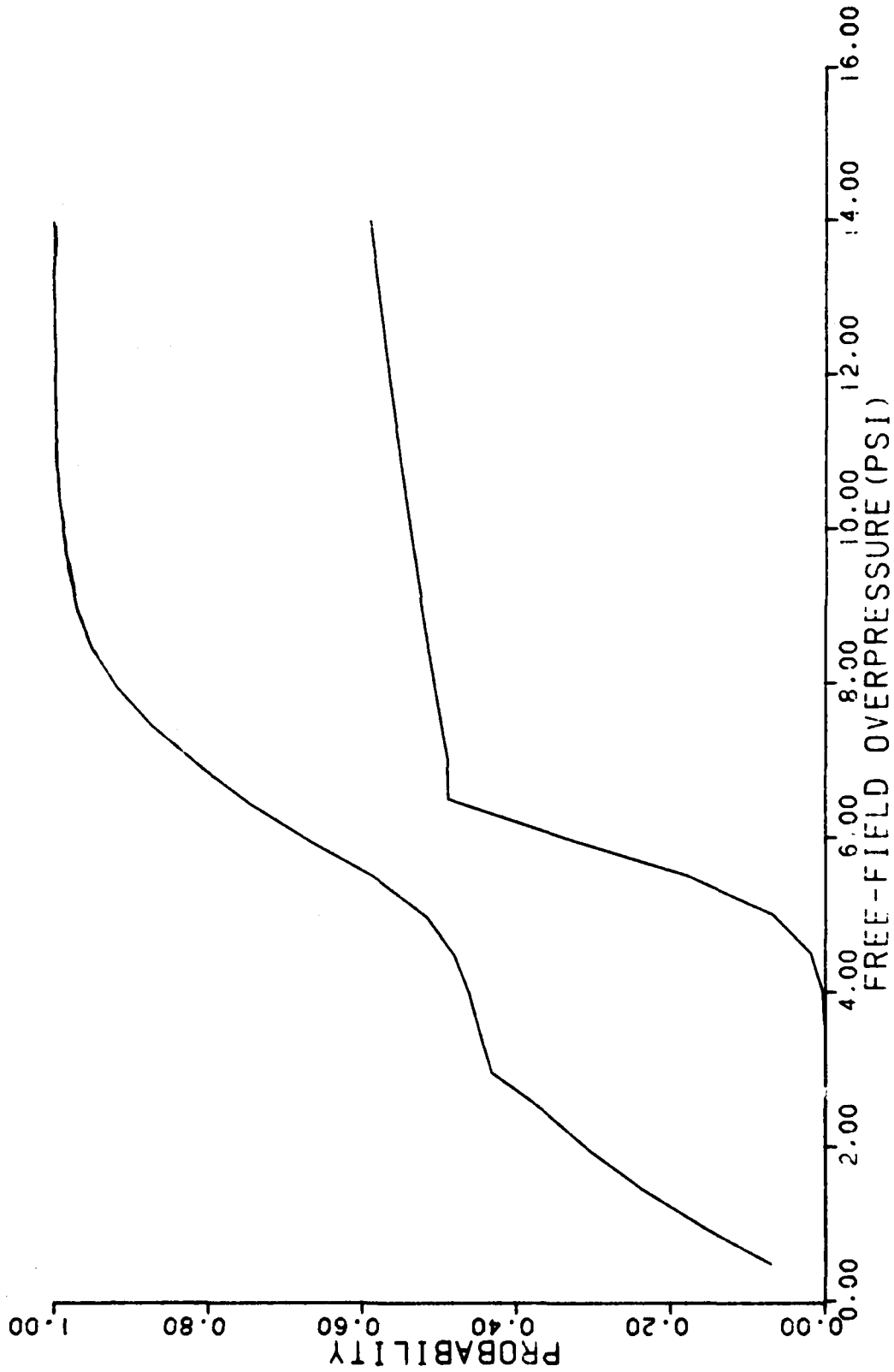


Figure C-53. Probability of slab failure (upper and lower bounds) case 6B.

CASE 6B3

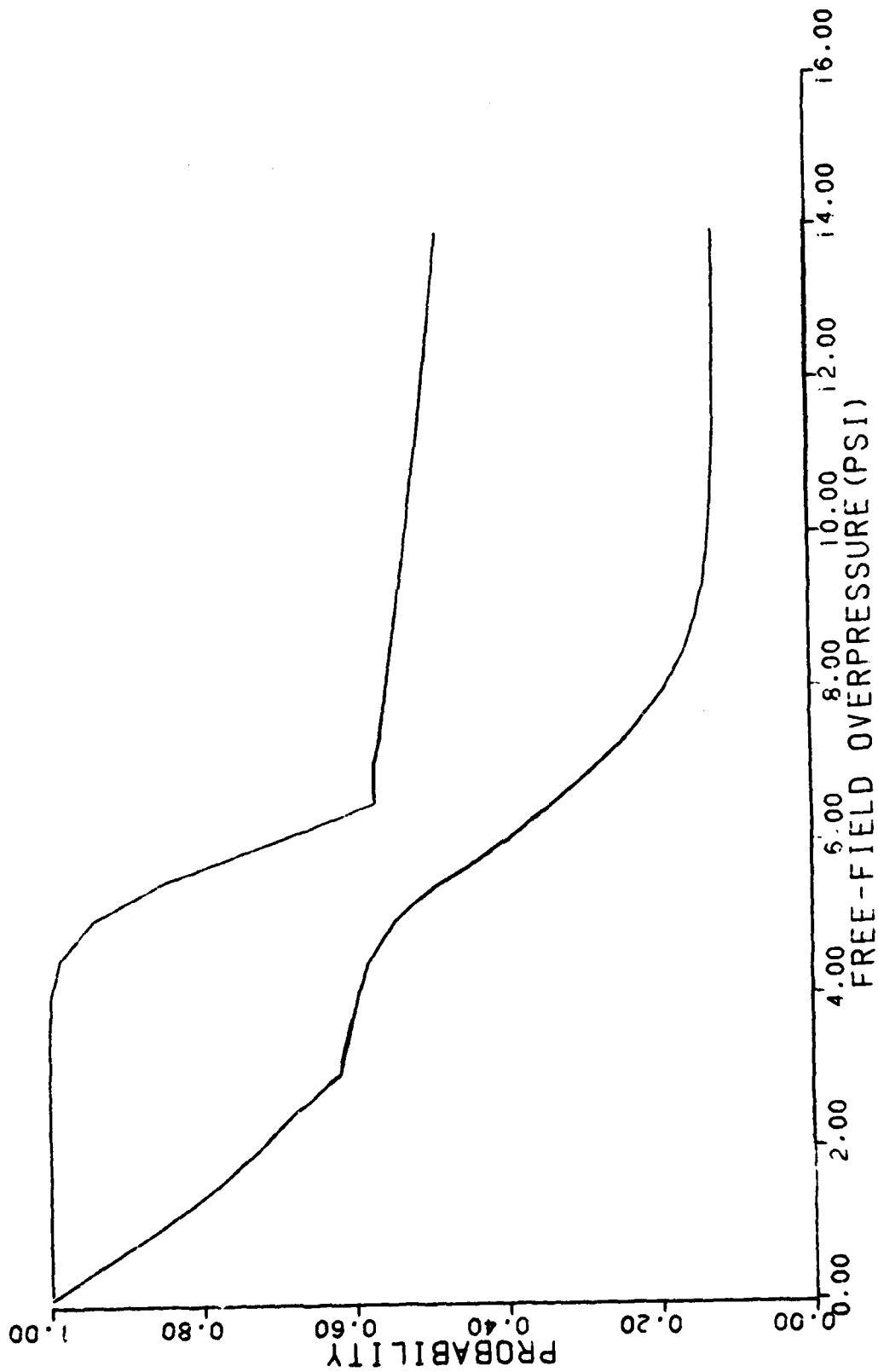


Figure C-54. Probability of people survival (upper and lower bounds) case 6B.

CASE 6C2

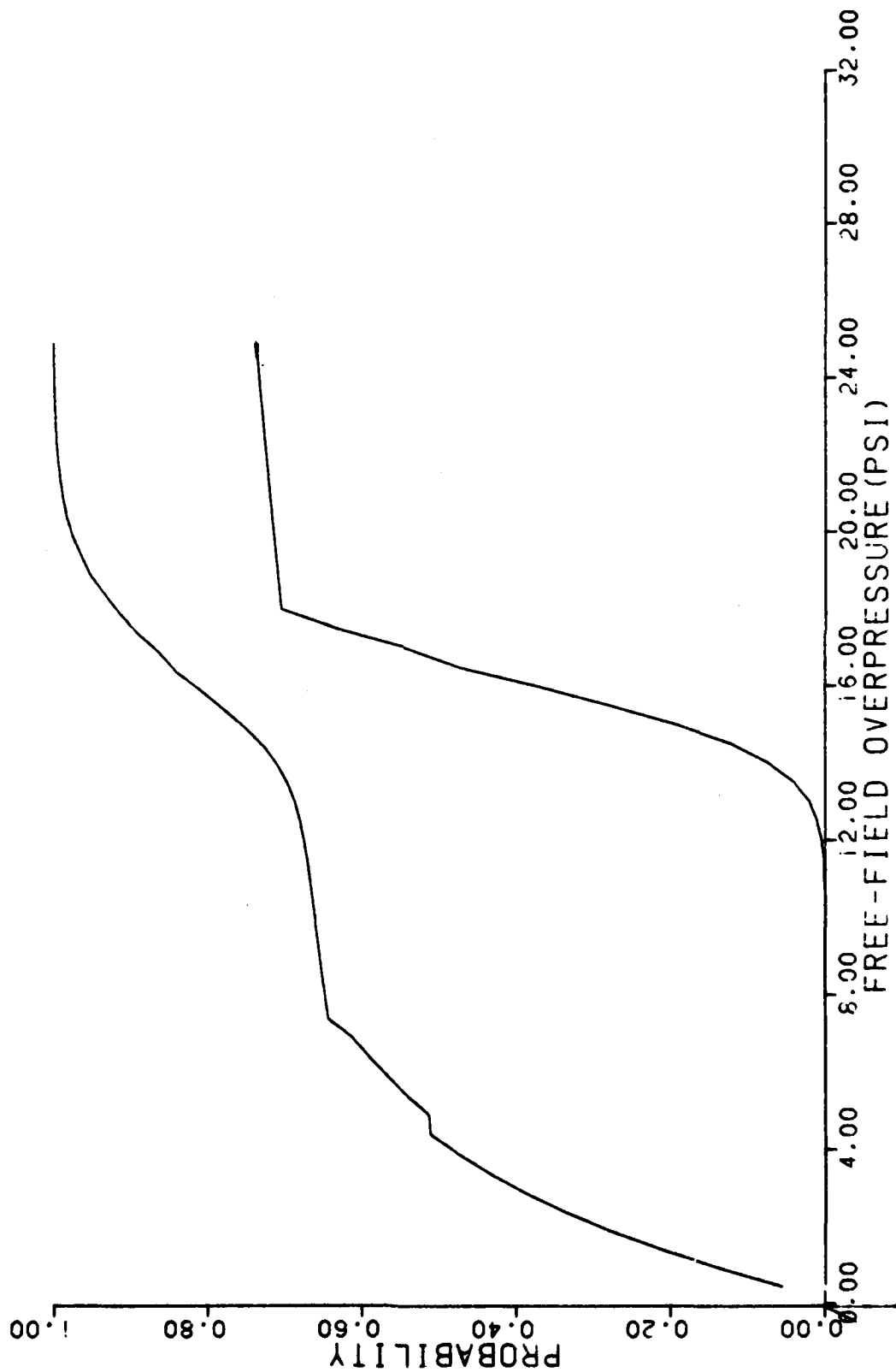


Figure C-55. Probability of slab failure (upper and lower bounds) case 6C.

CASE 6C3

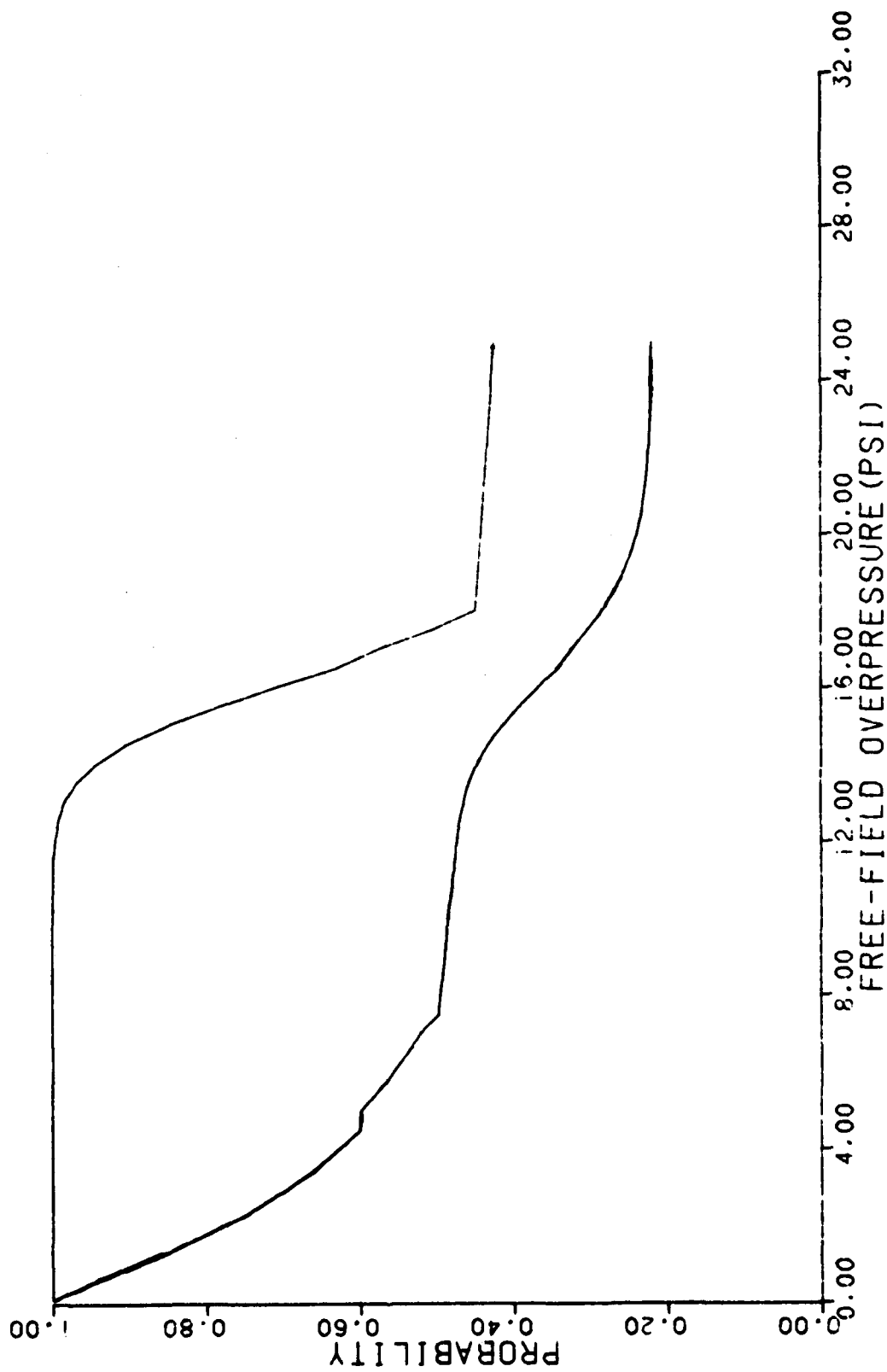


Figure C-56. Probability of people survival (upper and lower bounds) case 6C.

CASE 6D2

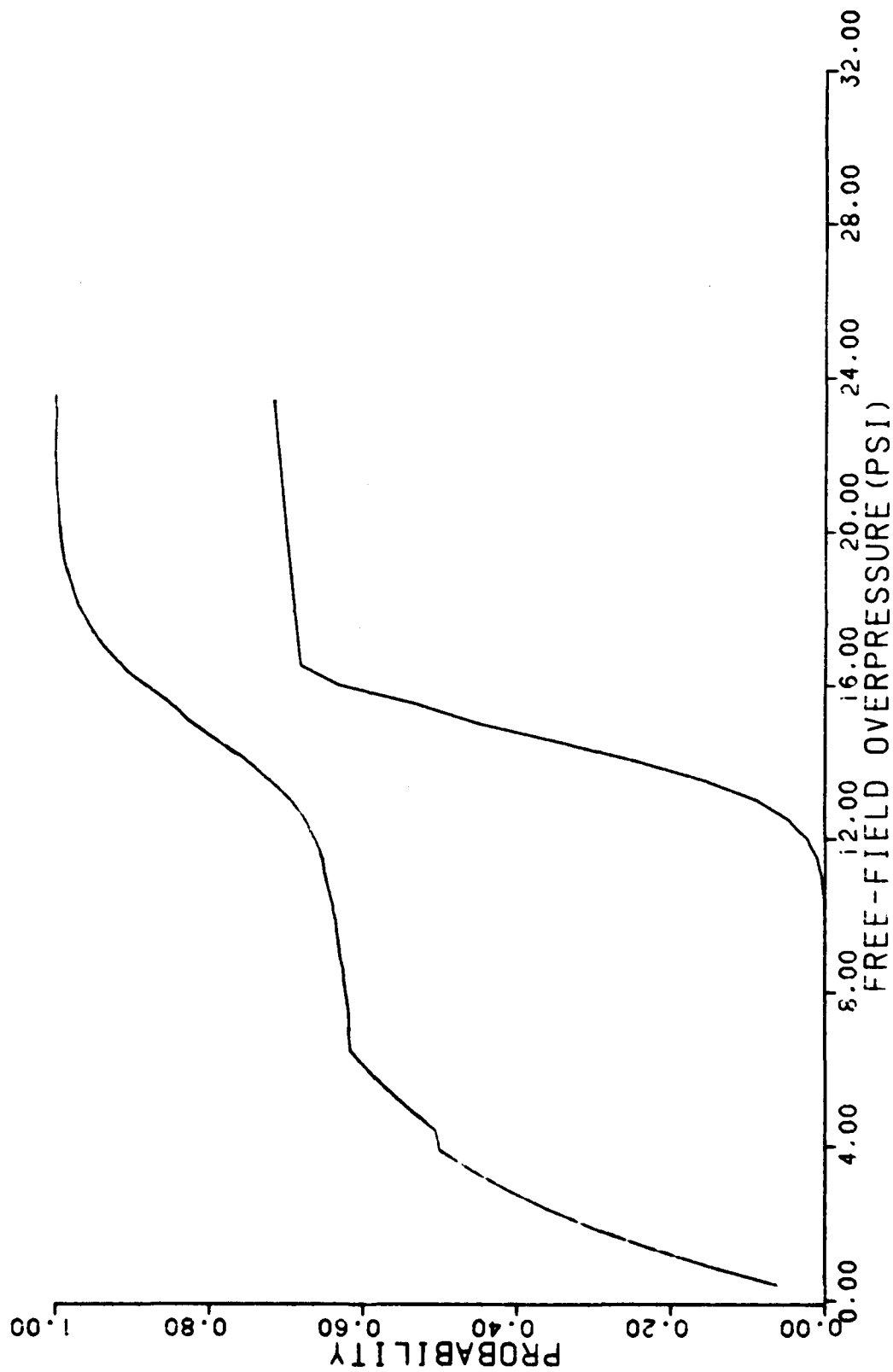


Figure C-57. Probability of slab failure (upper and lower bounds) case 6D.

CASE 6D3

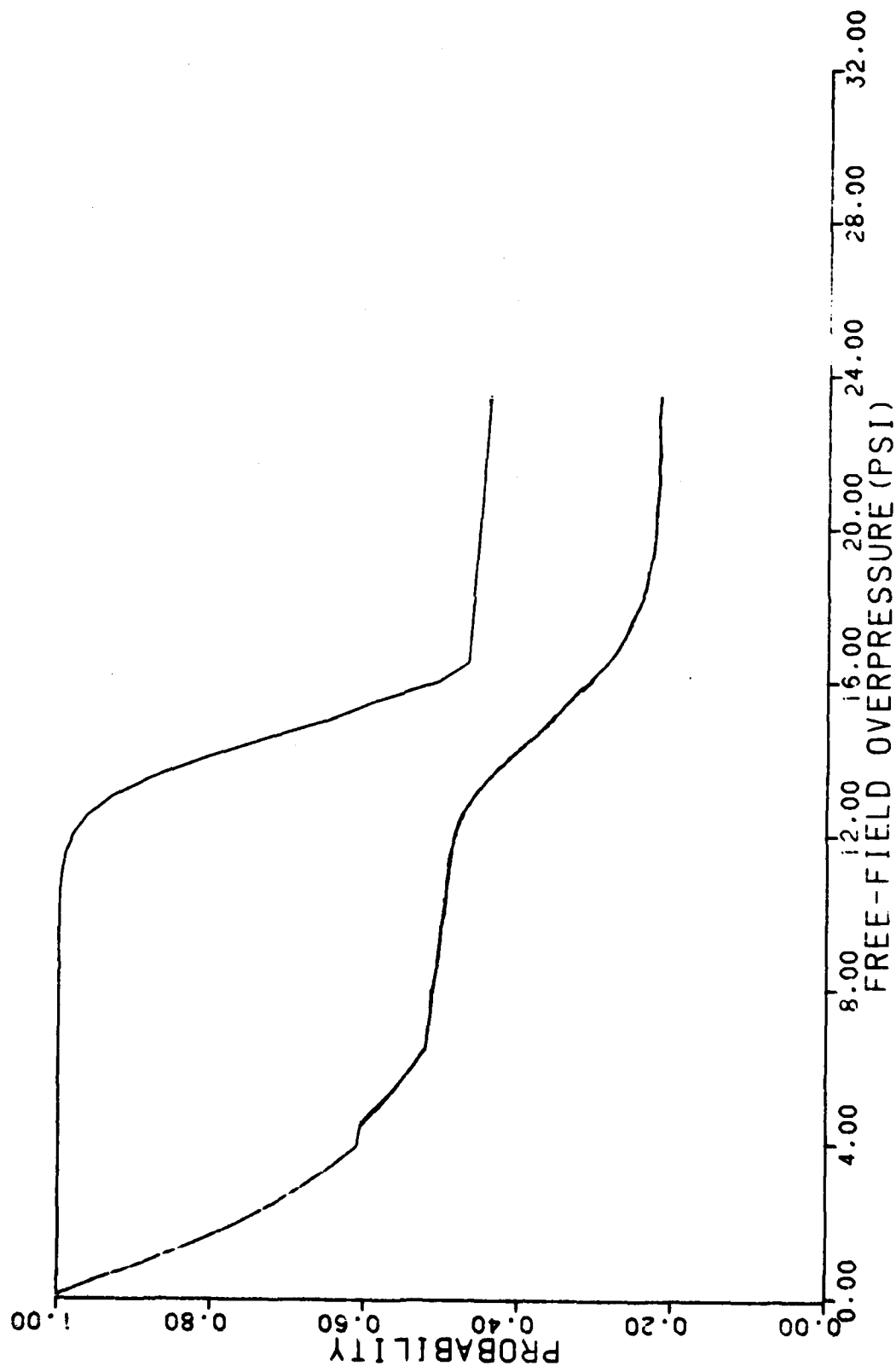


Figure C-58. Probability of people survival (upper and lower bounds) case 6D.

CASE 6E2

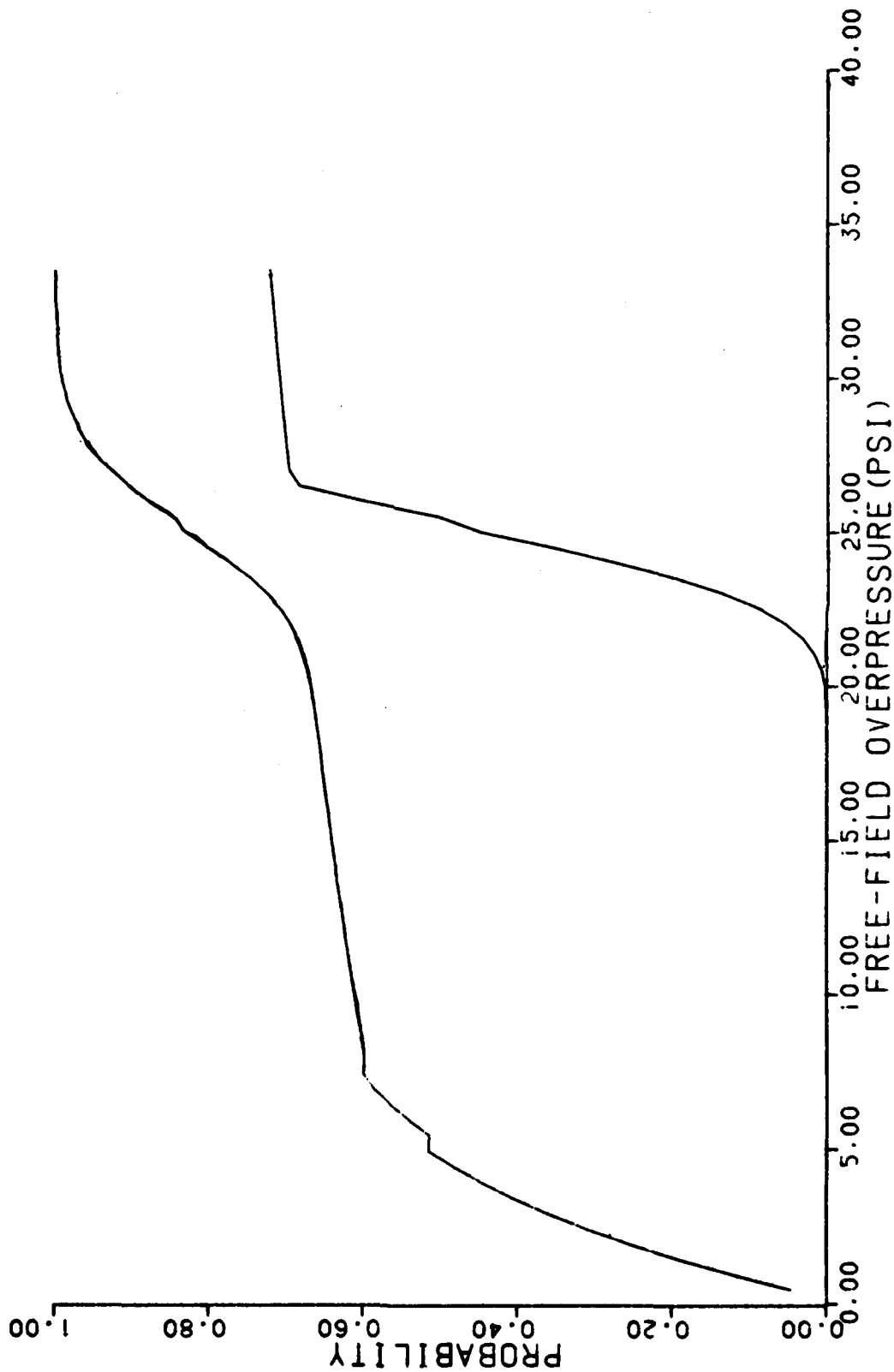


Figure C-59. Probability of slab failure (upper and lower bounds) case 6E.

CASE 6E3

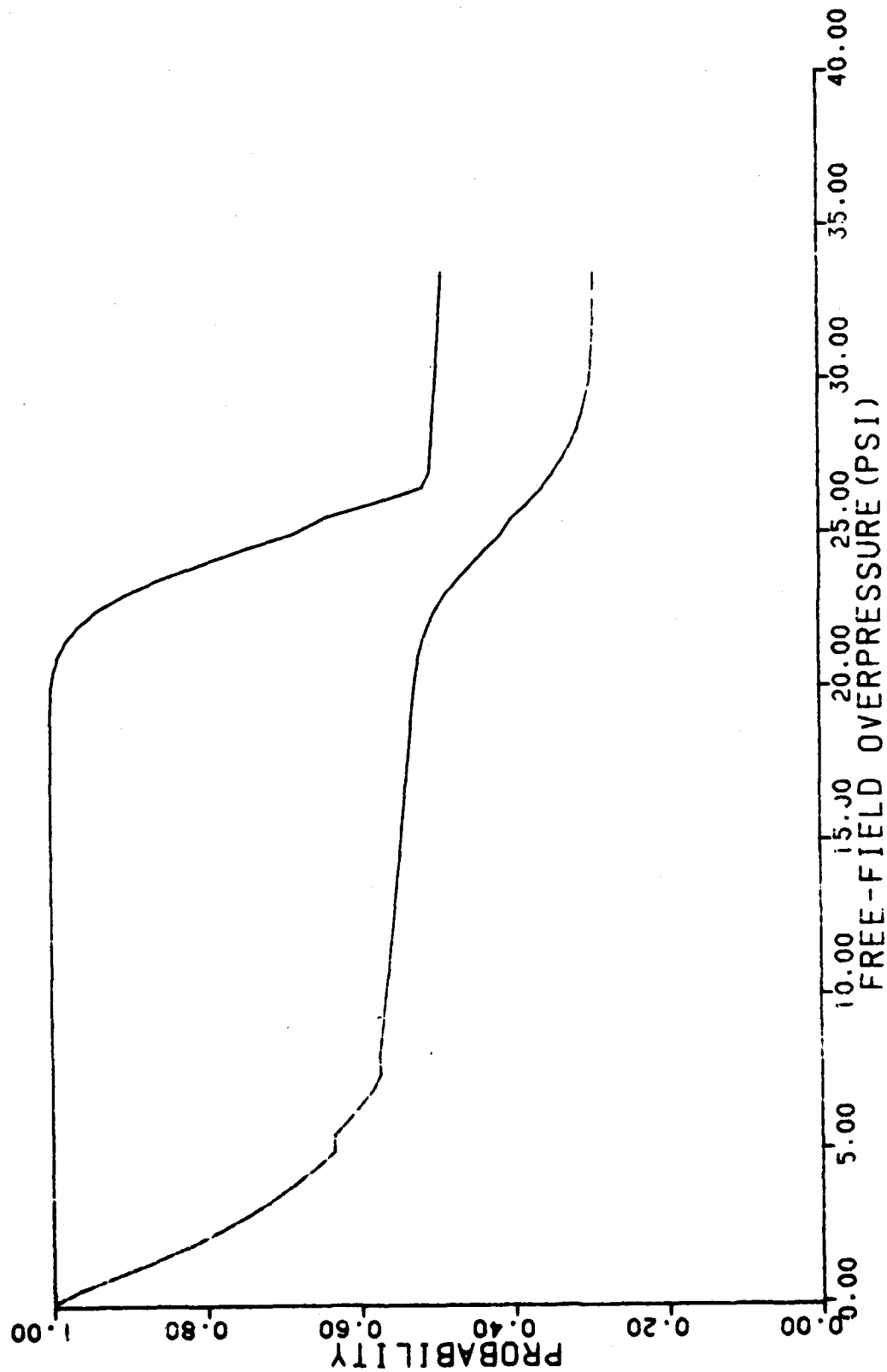


Figure C-60. Probability of people survival (upper and lower bounds) case 6E.

CASE 7A2

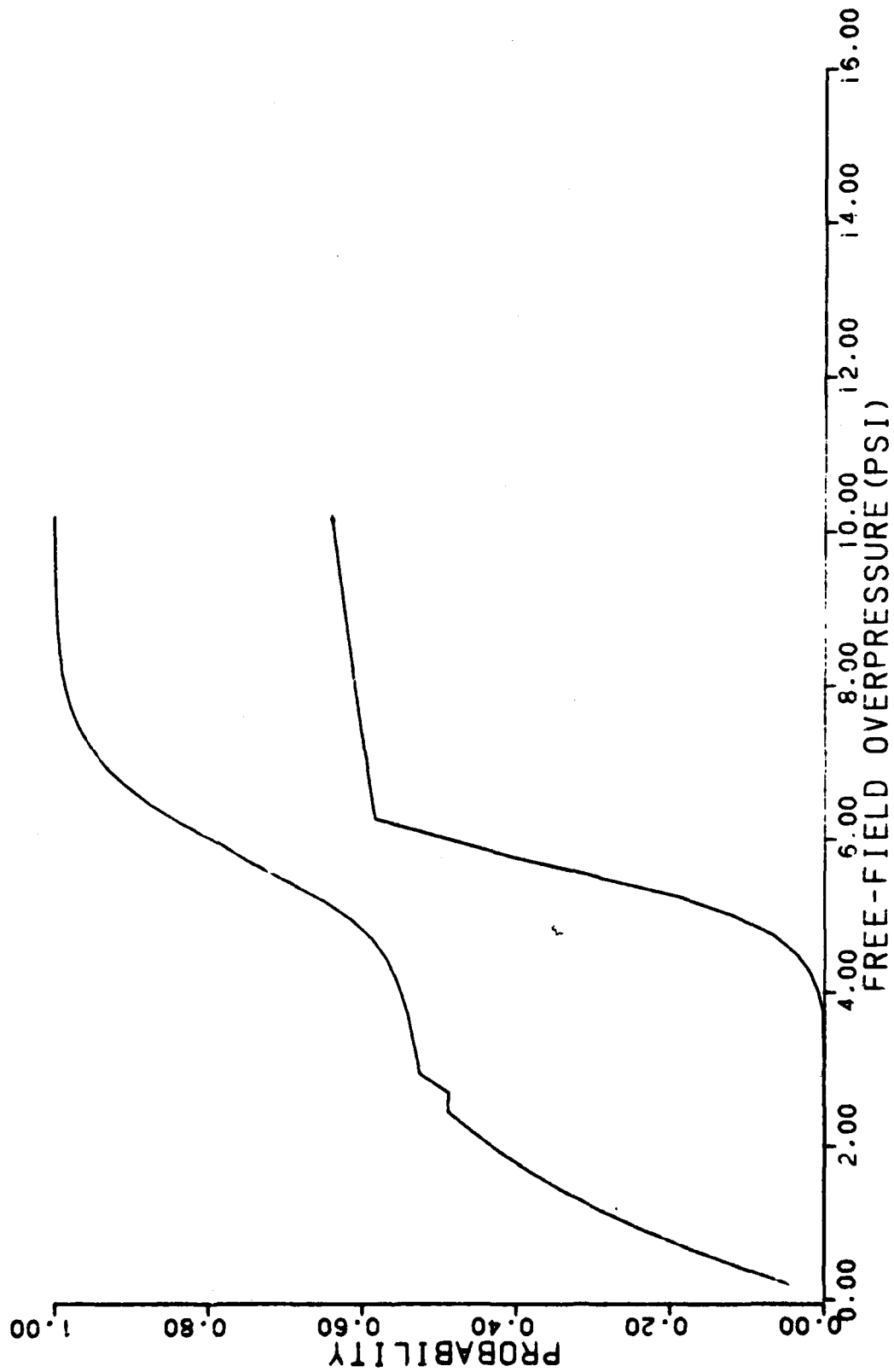


Figure C-61. Probability of slab failure (upper and lower bounds) case 7A.

CASE 7A3

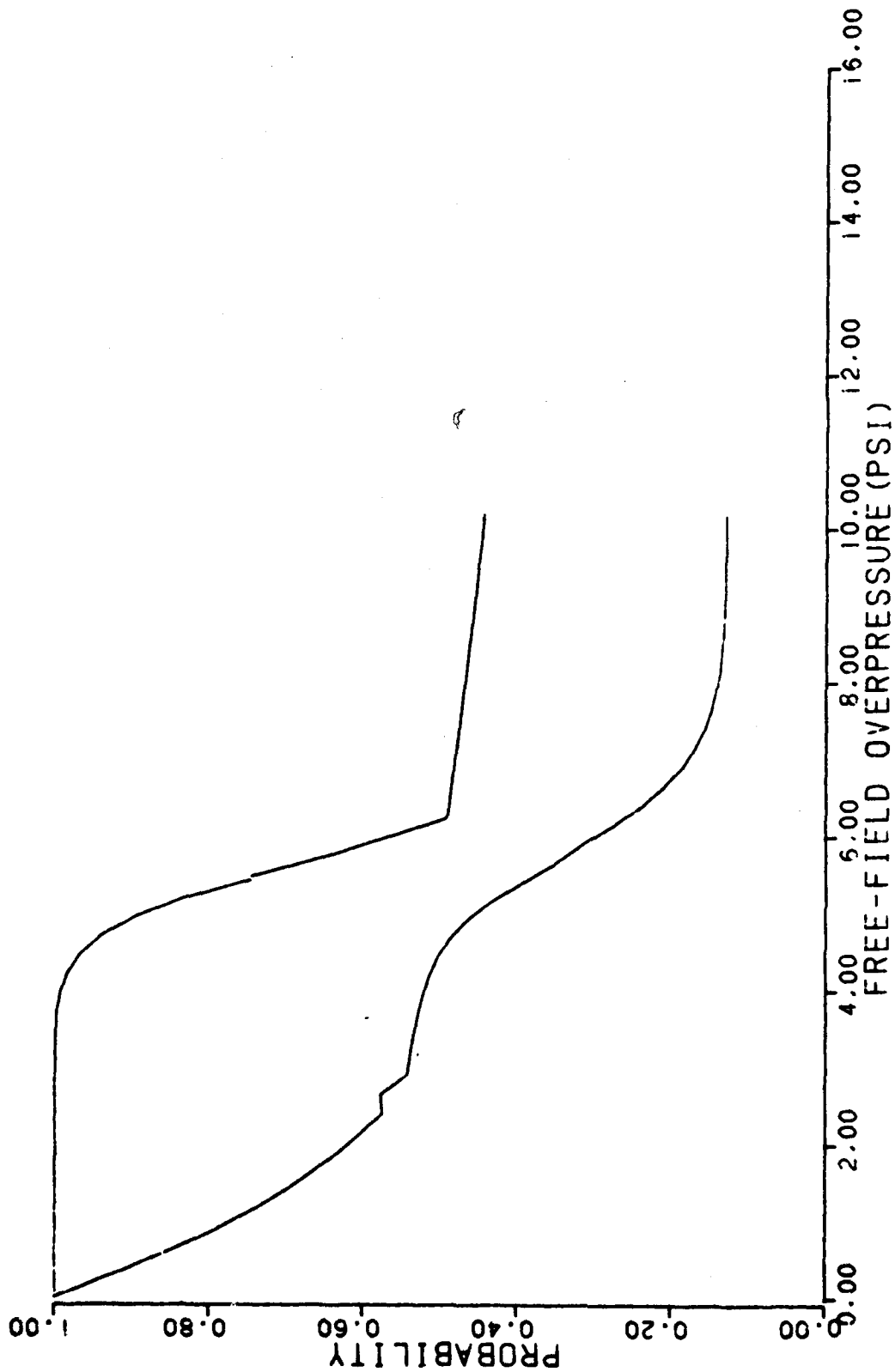


Figure C-62. Probability of people survival (upper and lower bounds) case 7A.

CASE 7B2

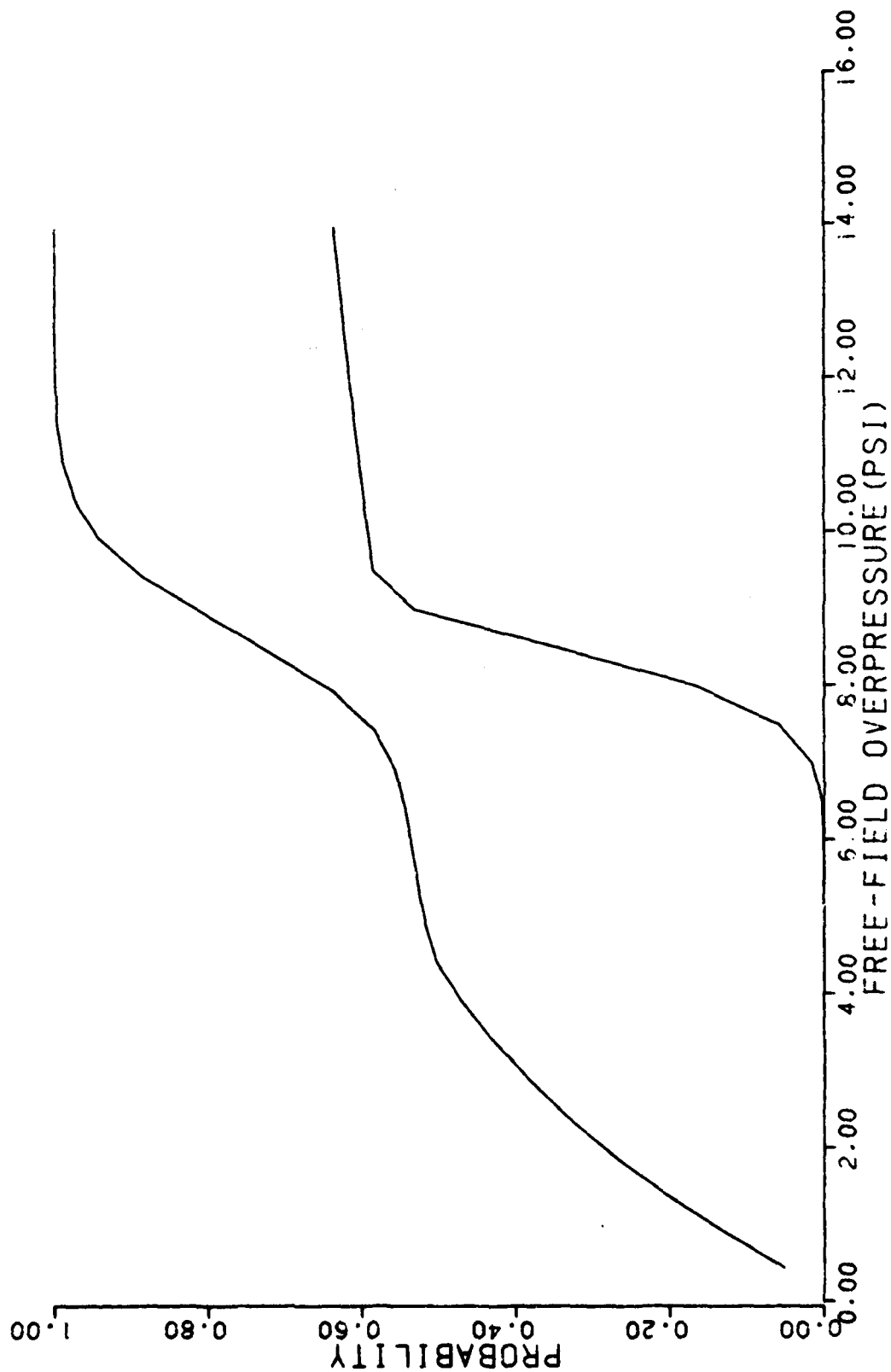


Figure C-63. Probability of slab failure (upper and lower bounds) case 7B.

CASE 7B3

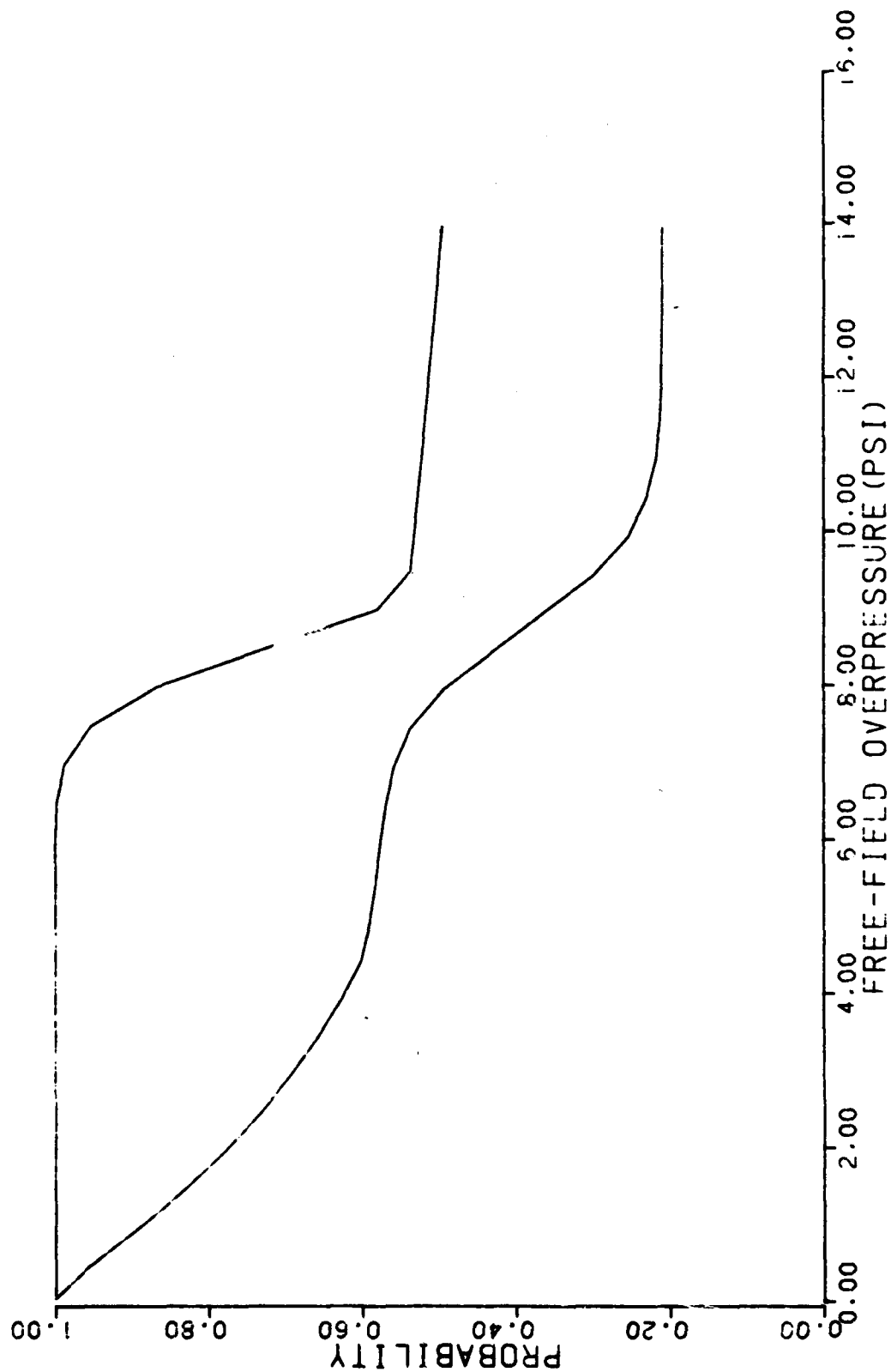


Figure C-64. Probability of people survival (upper and lower bounds) case 7B.

CASE 7C2

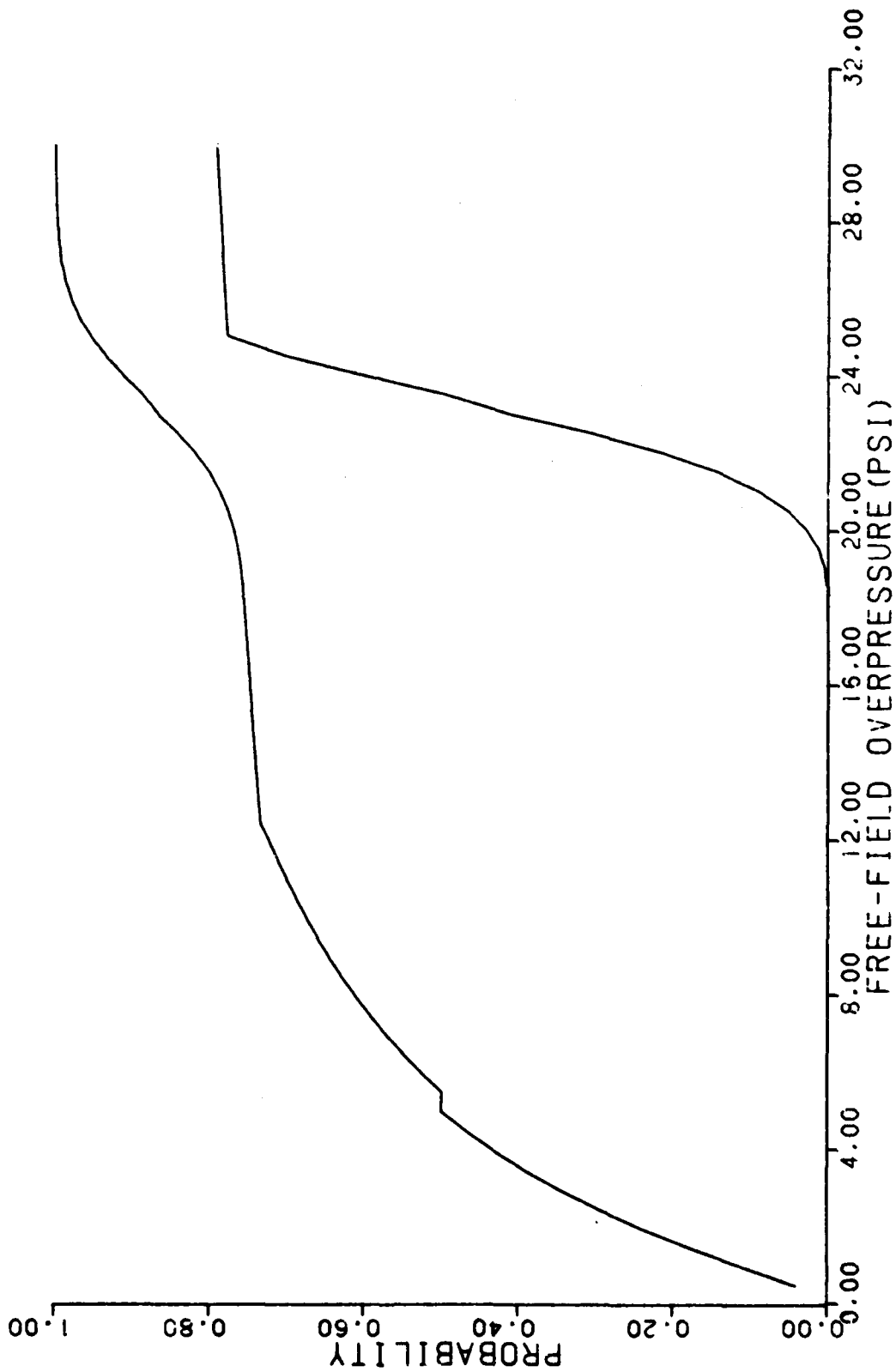


Figure C-65. Probability of slab failure (upper and lower bounds) case 7C.

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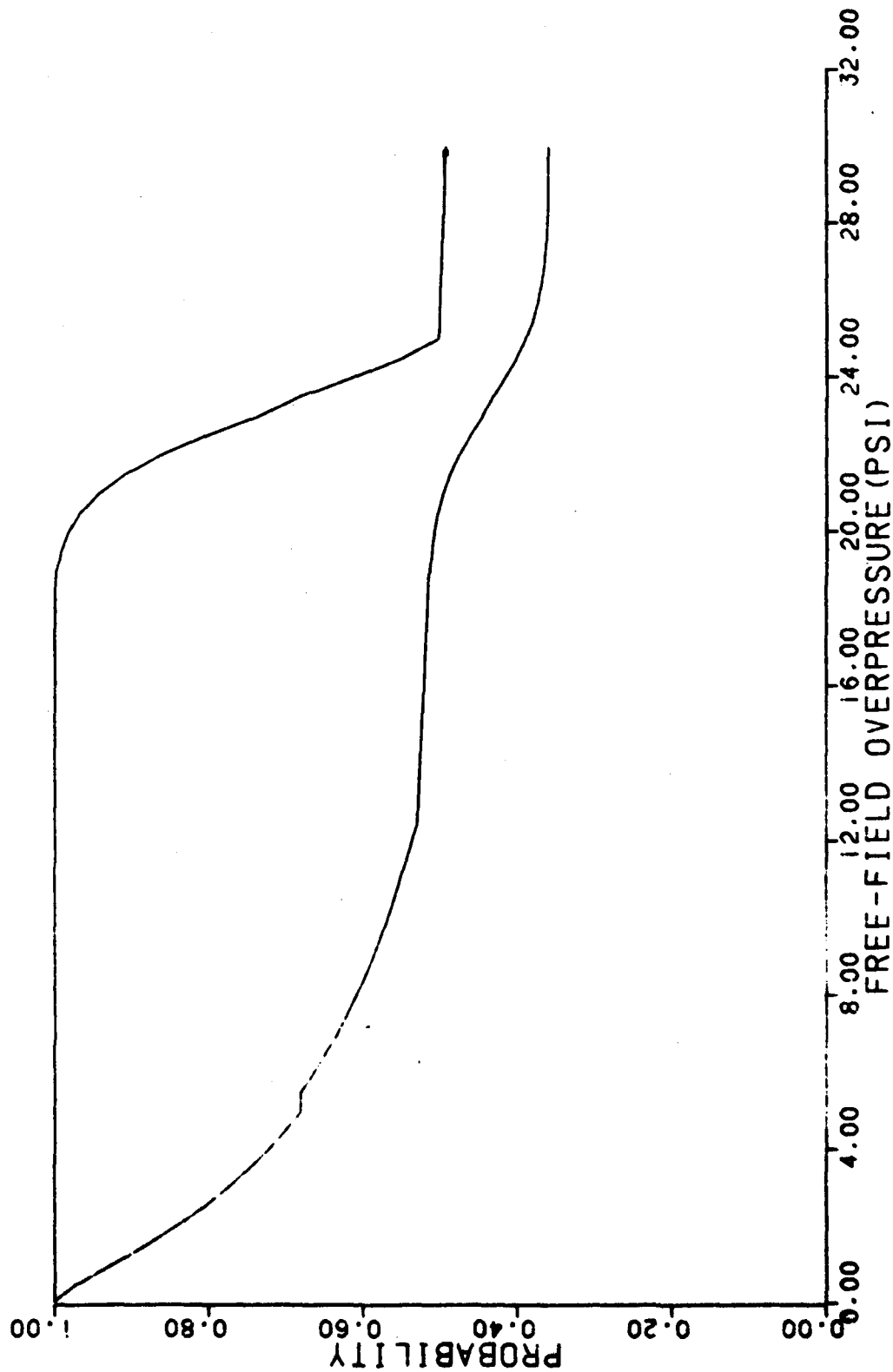


Figure C-66. Probability of people survival (upper and lower bounds) case 7C.

CASE 7D2

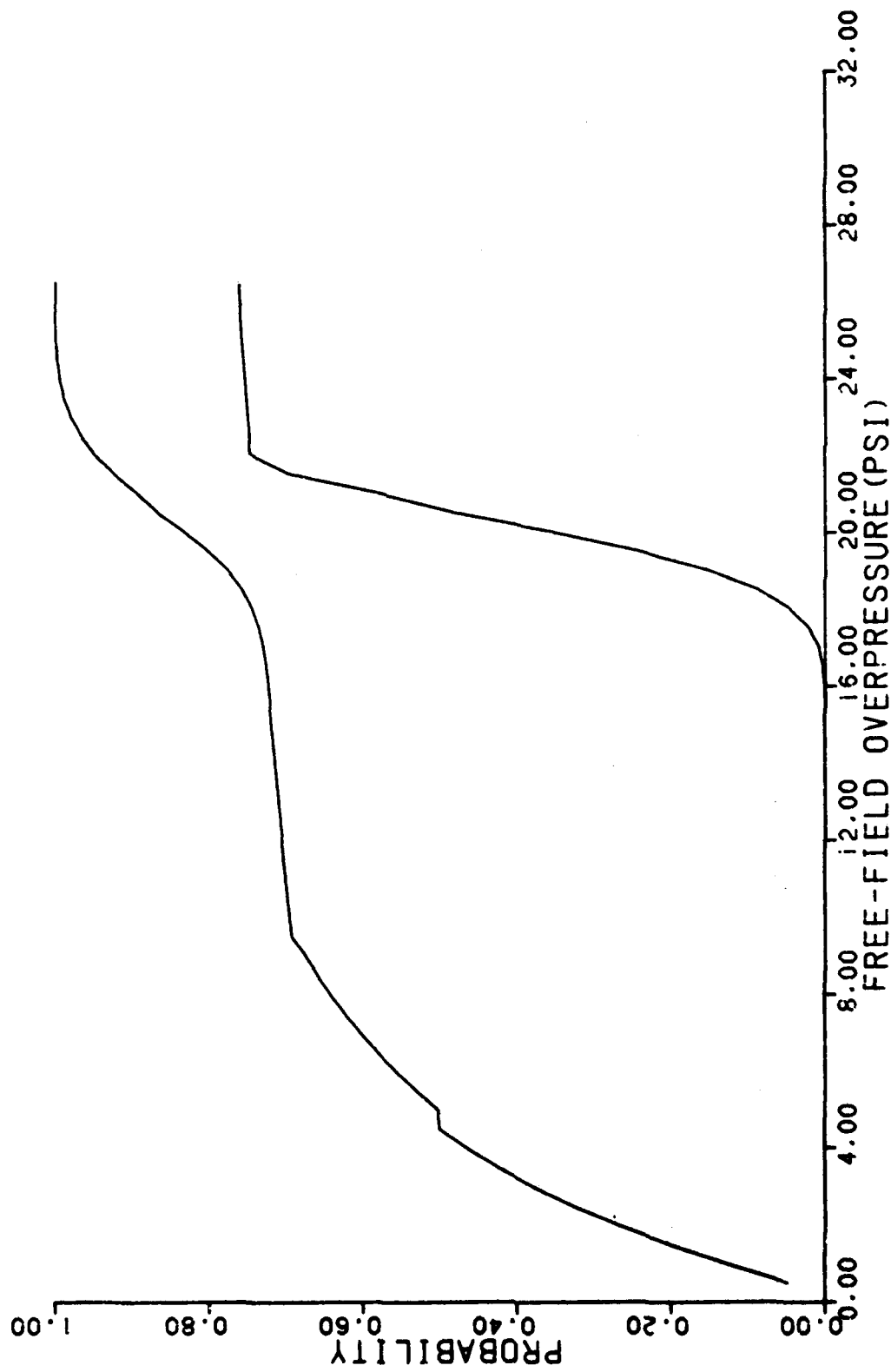


Figure C-67. Probability of slab failure (upper and lower bounds) case 7D.

CASE 7D3

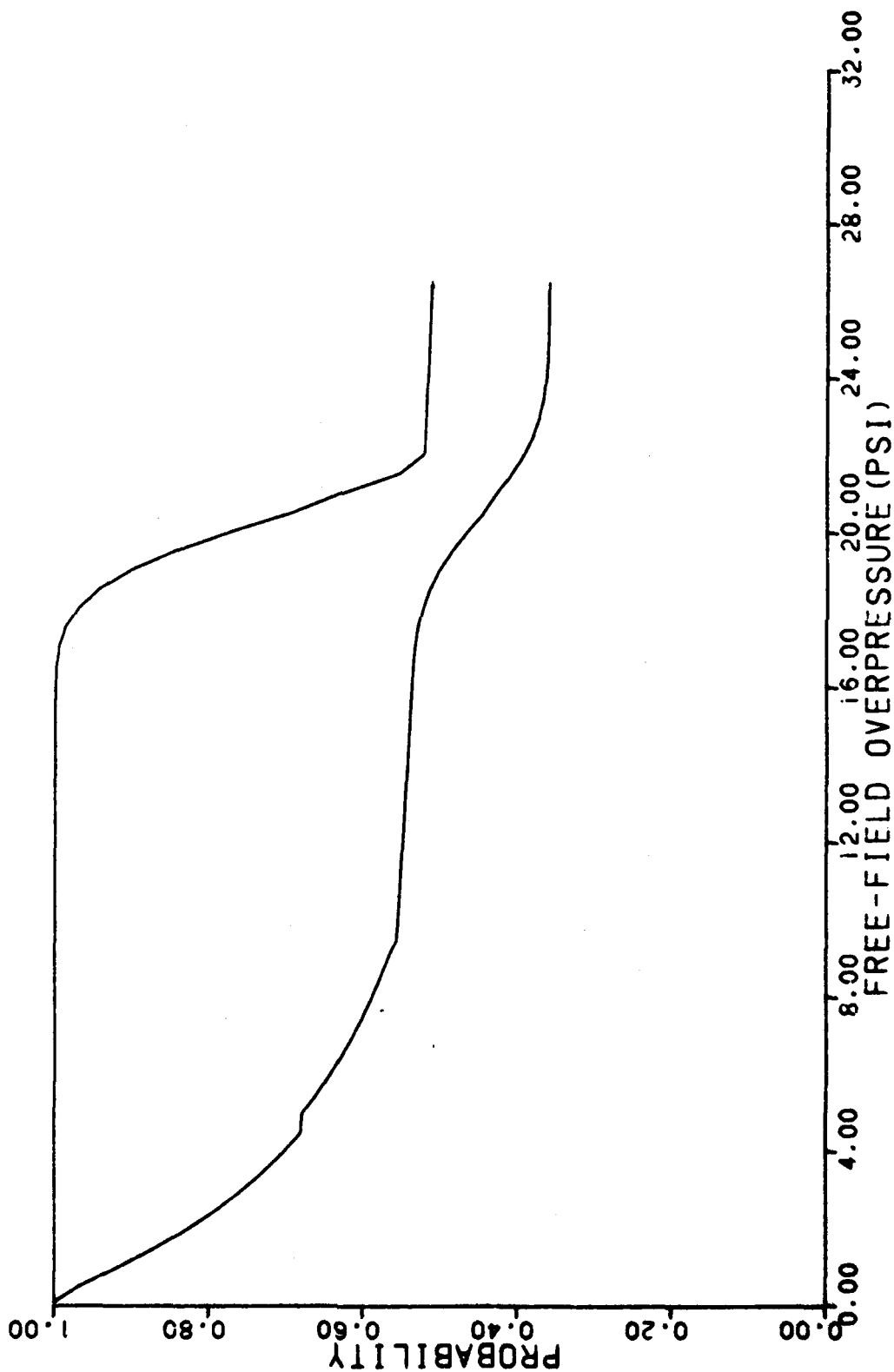


Figure C-68. Probability of people survival (upper and lower bounds) case 7D.

CASE 7E2

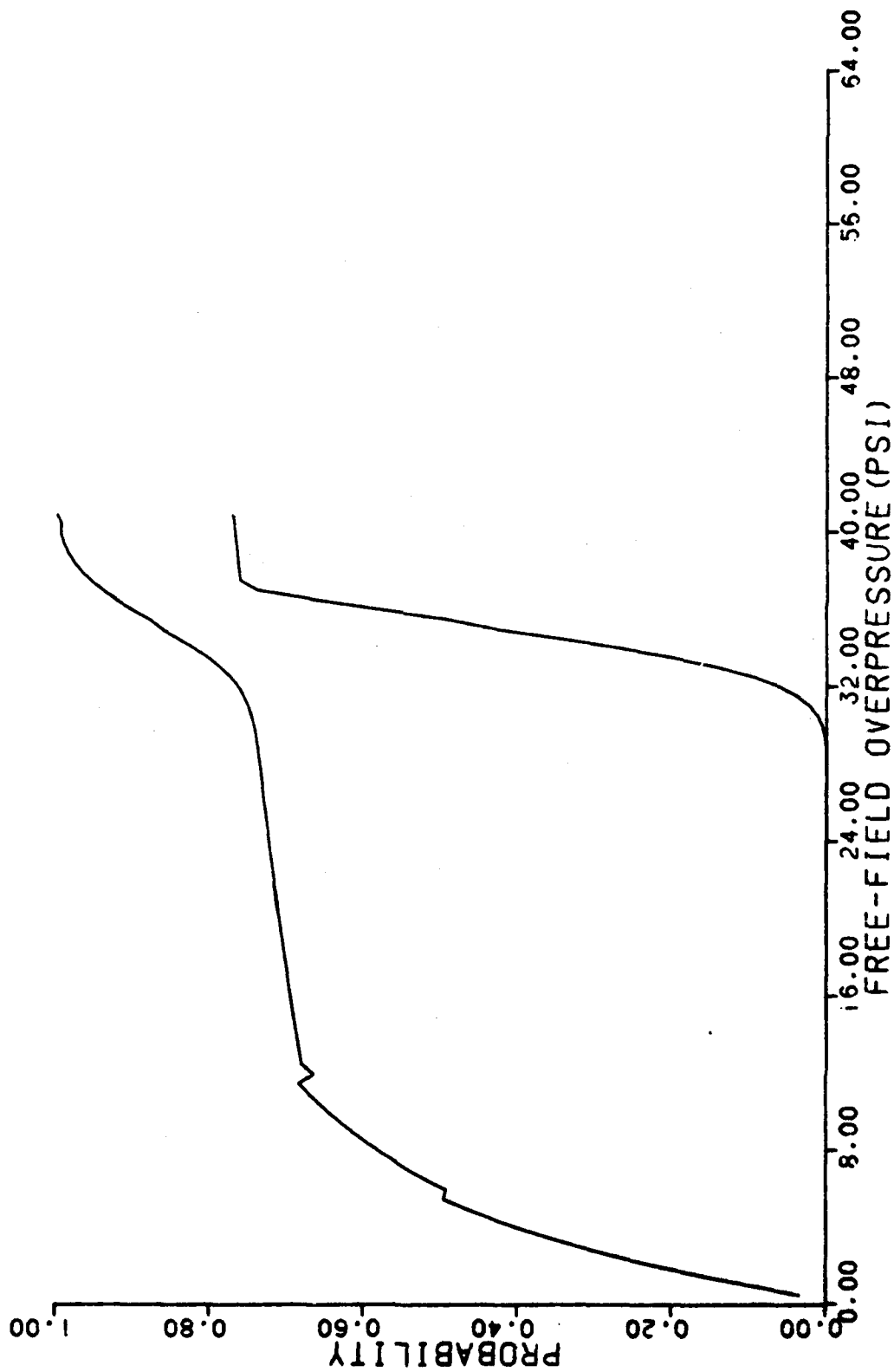


Figure C-69. Probability of slab failure (upper and lower bounds) case 7E.

CASE 7E3

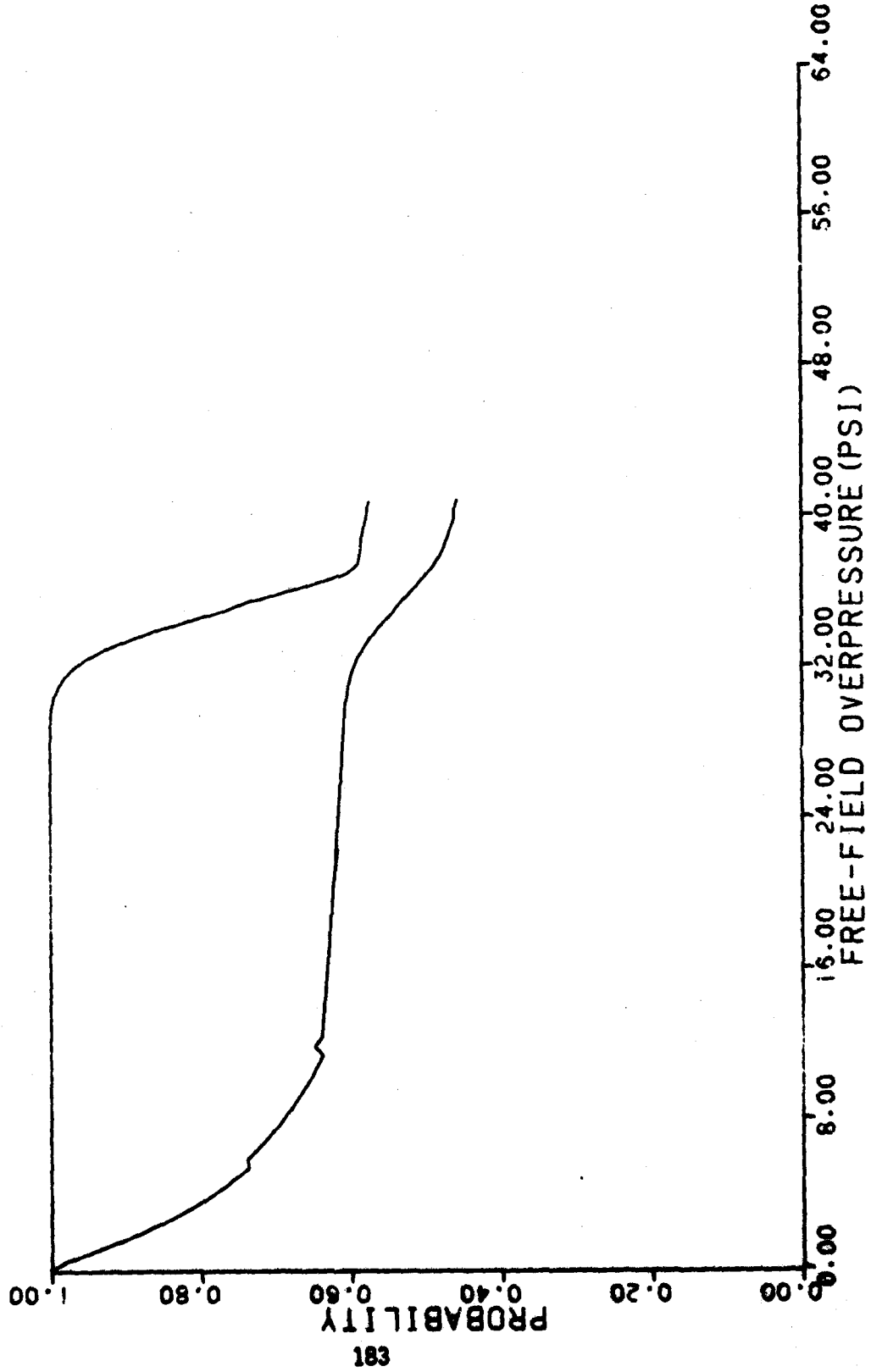


Figure C-70. Probability of people survival (upper and lower bounds) case 7E.

CASE 8A2

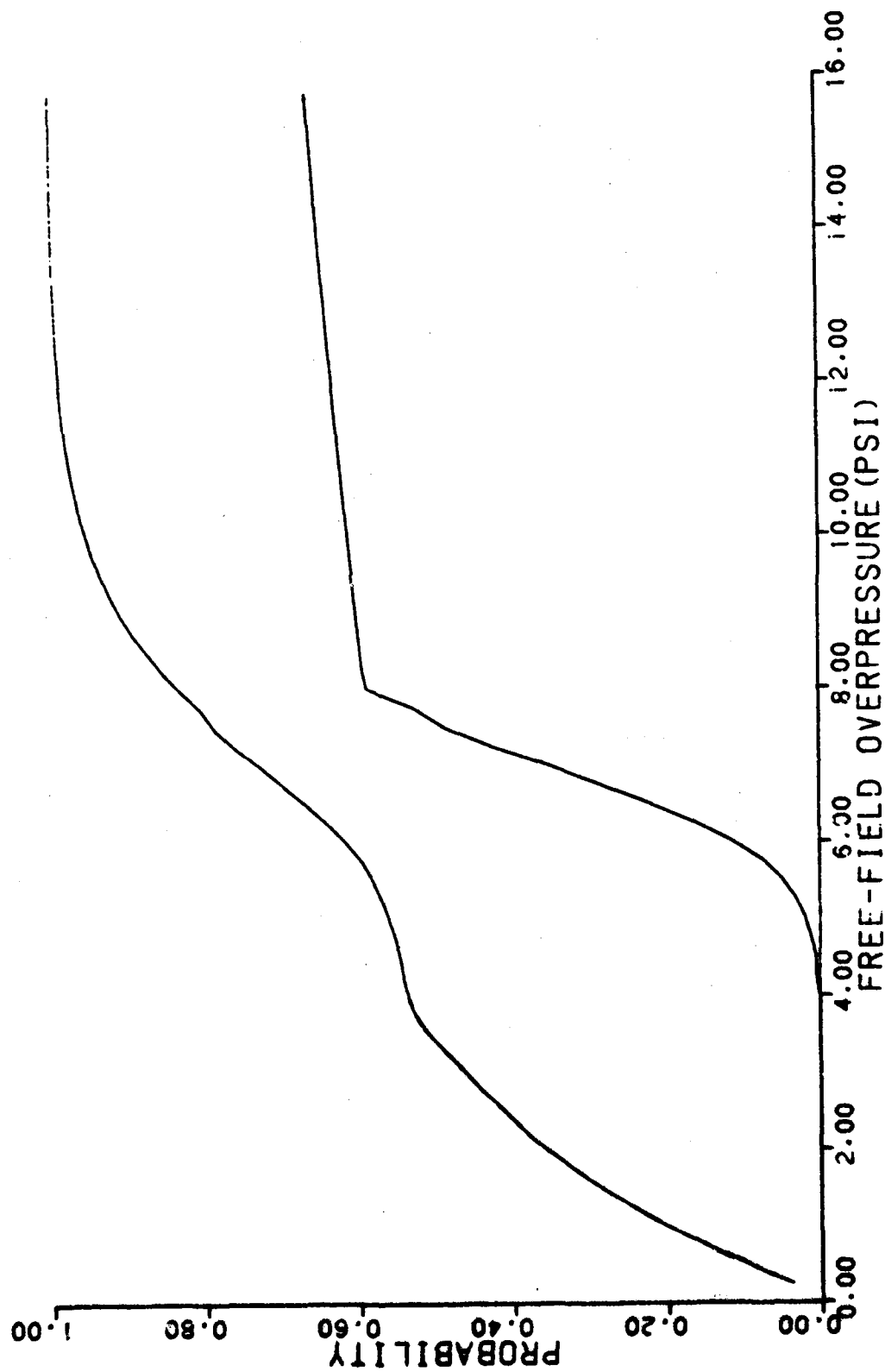


Figure C-71. Probability of slab failure (upper and lower bounds) case 8A.

CASE 8A3

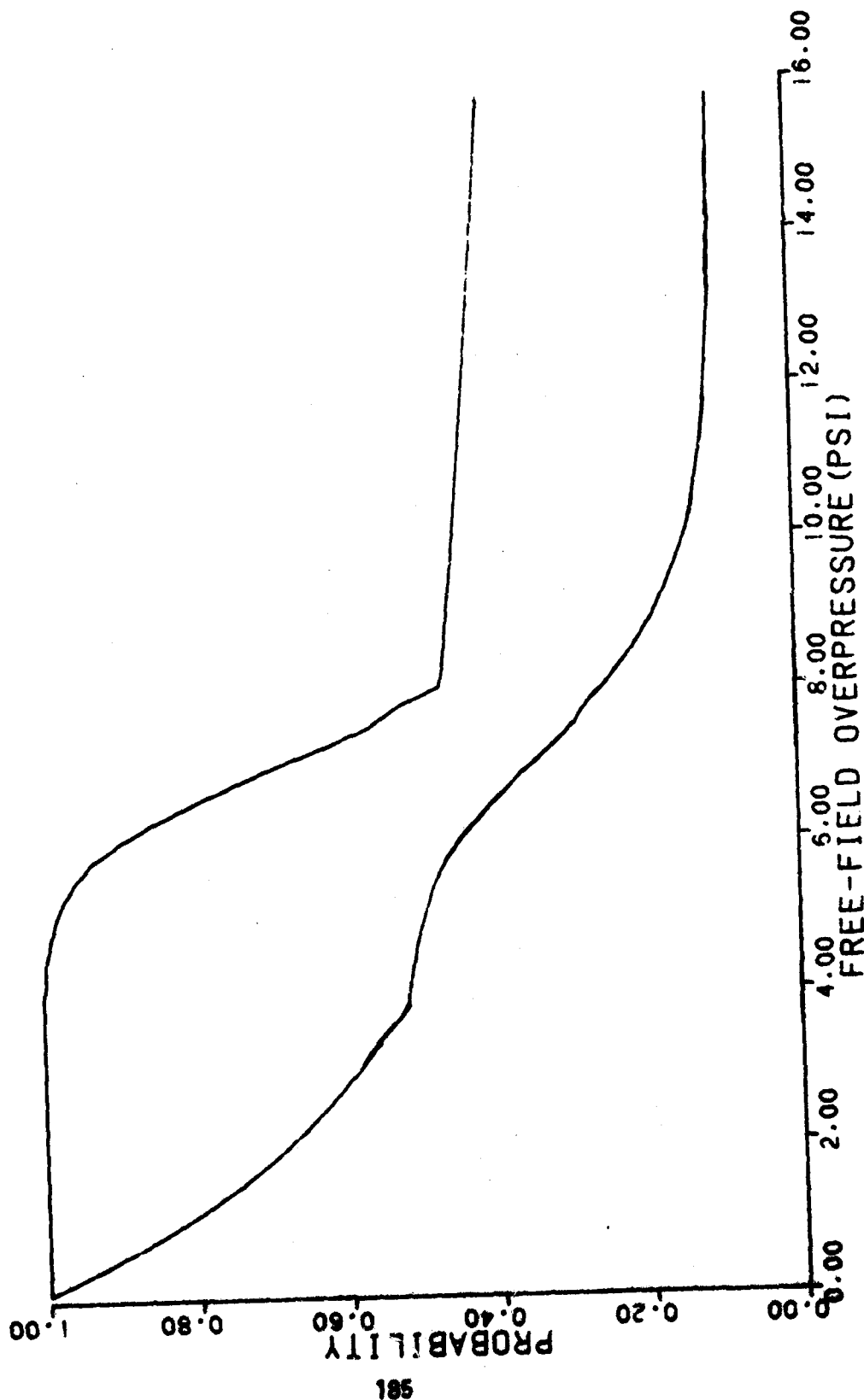


Figure C-72. Probability of people survival (upper and lower bounds) case 8A.

CASE 8B2

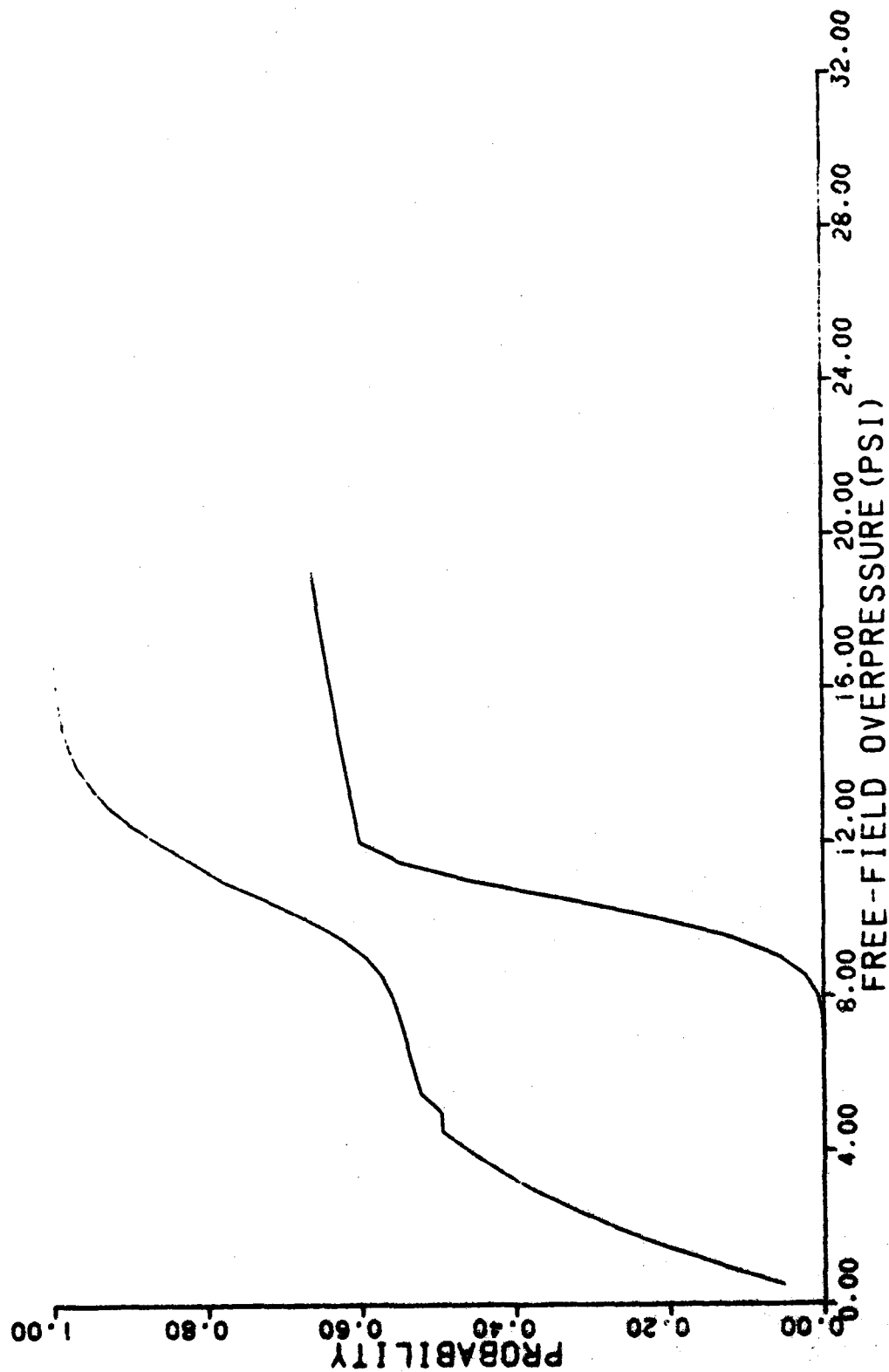


Figure C-73. Probability of slab failure (upper and lower bounds) case 8B.

CASE 8B3

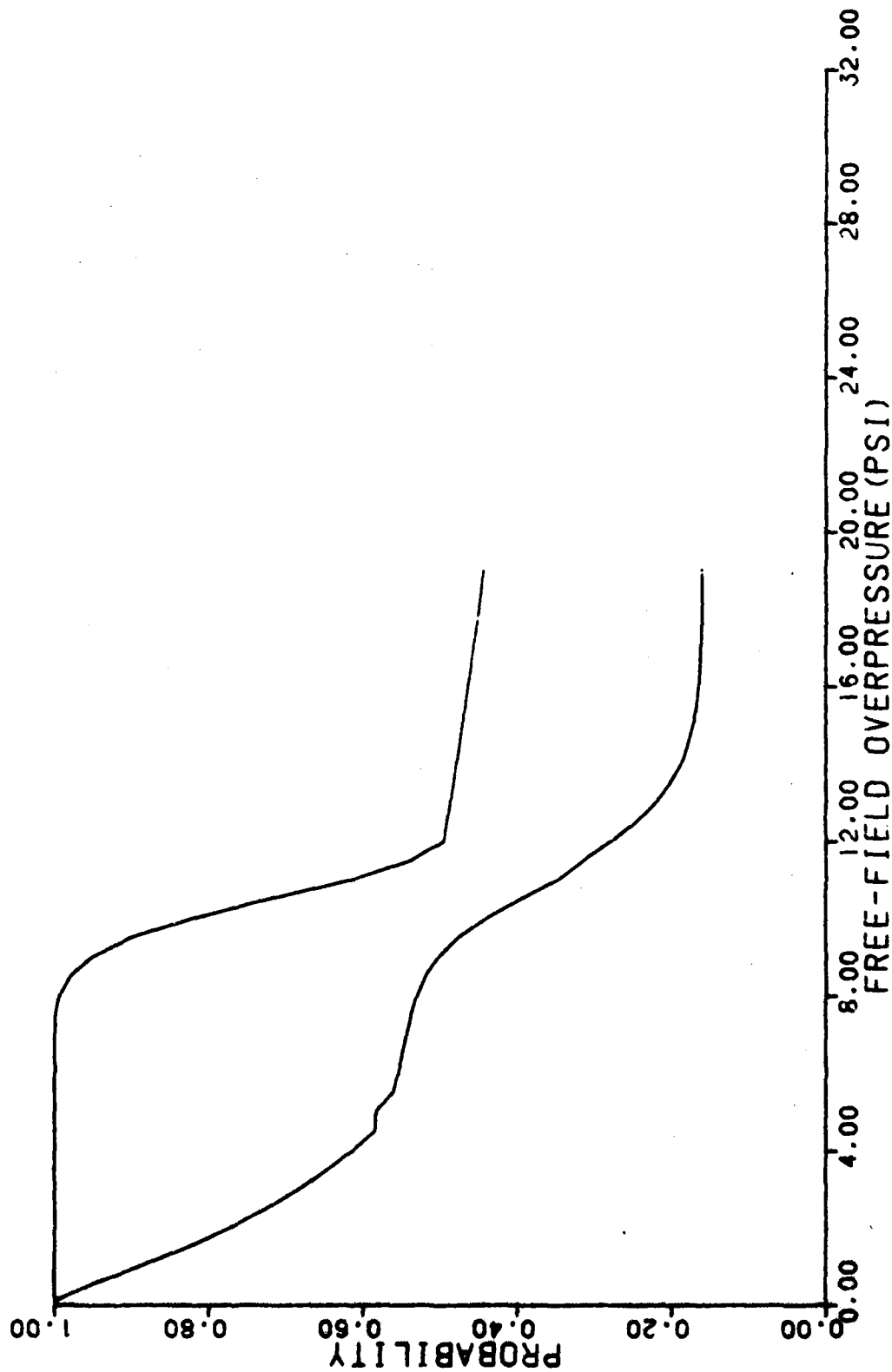


Figure C-74. Probability of people survival (upper and lower bounds) case 88.

CASE 8C2



Figure C-75. Probability of slab failure (upper and lower bounds) case 8C.

CASE 8C3

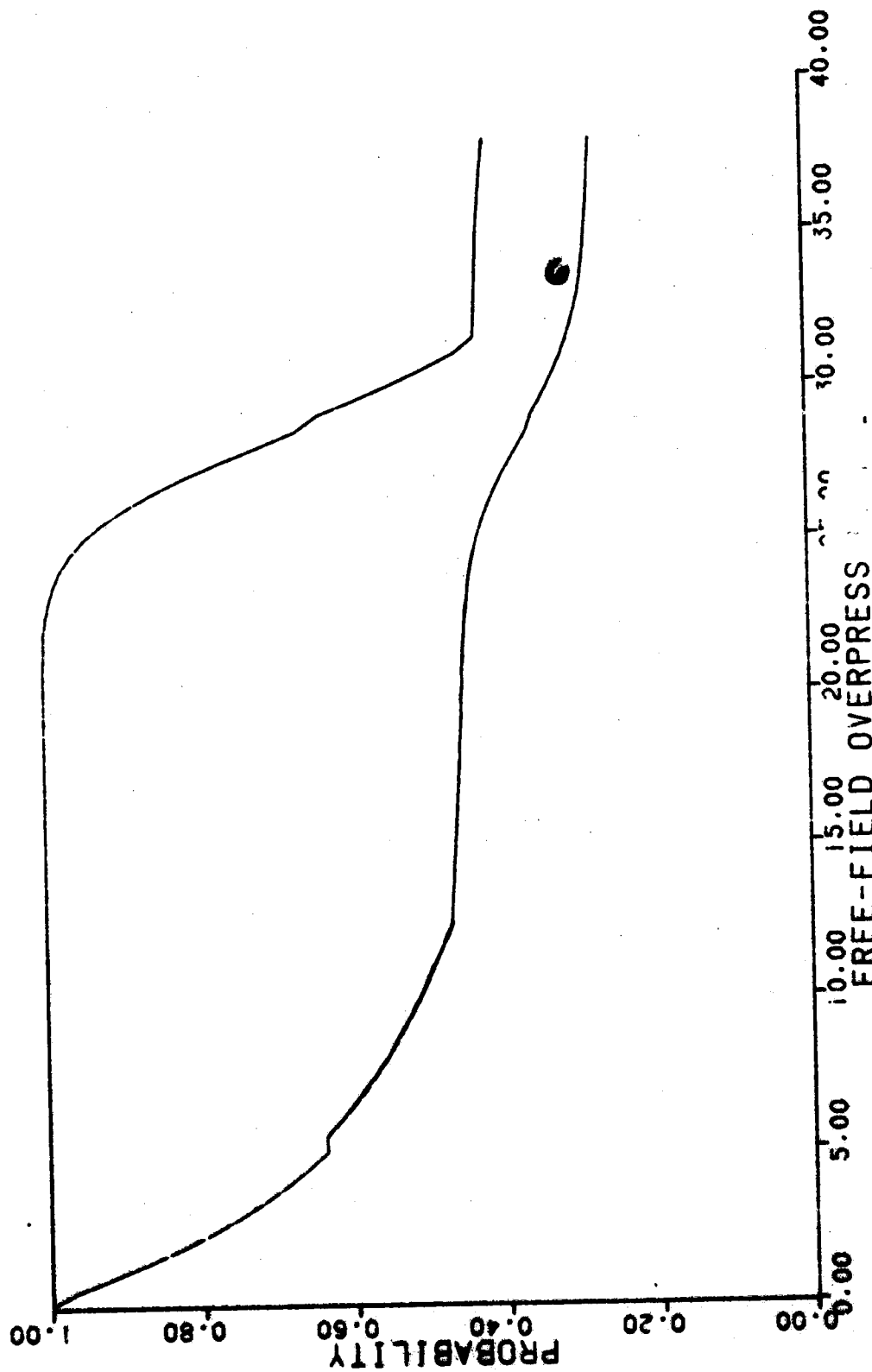


Figure C-76. Probability of people survival (upper and lower bounds) case 8C.

CASE 8D2

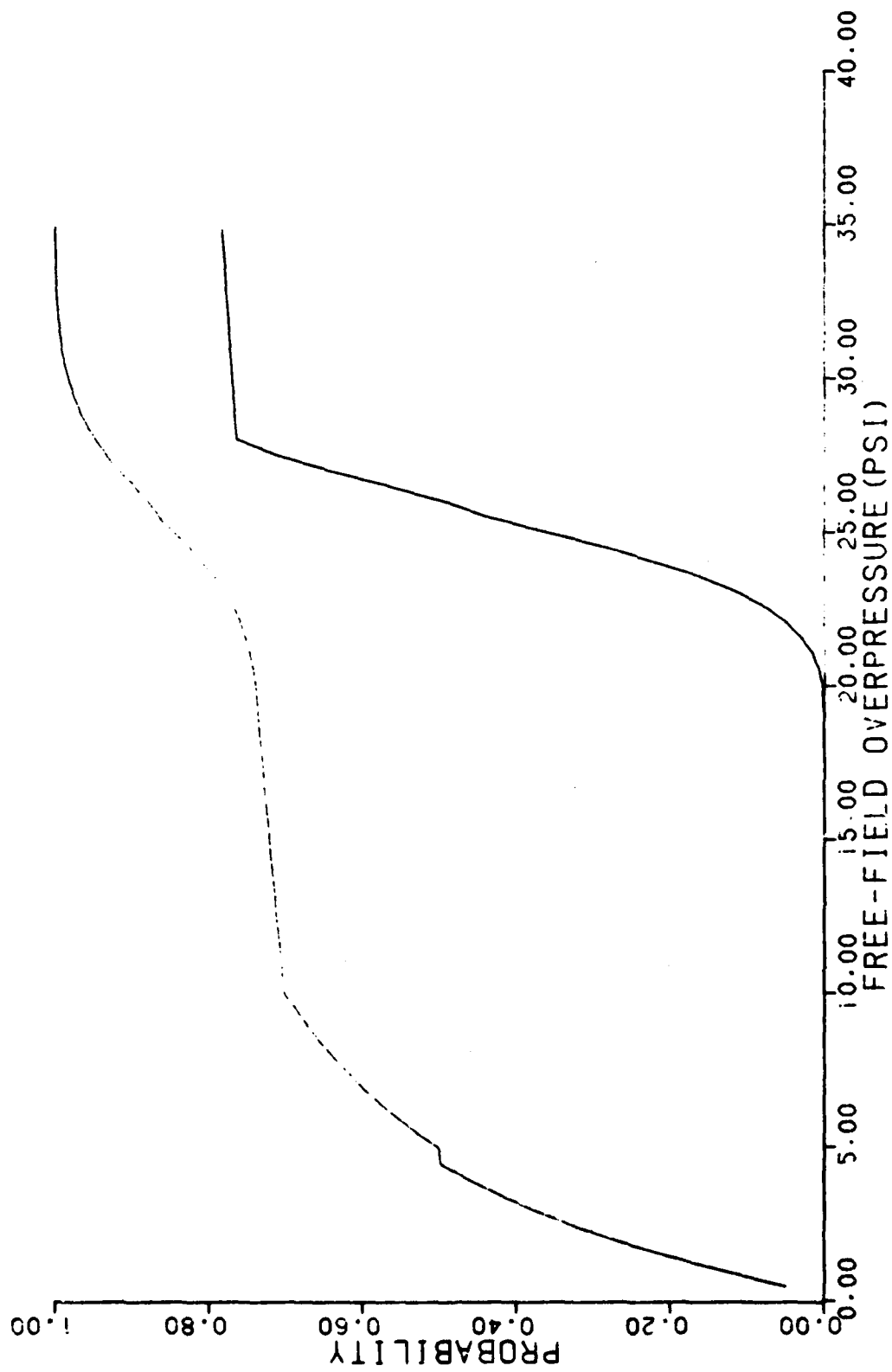


Figure C-77. Probability of slab failure (upper and lower bounds) case 8D.

CASE 8D3

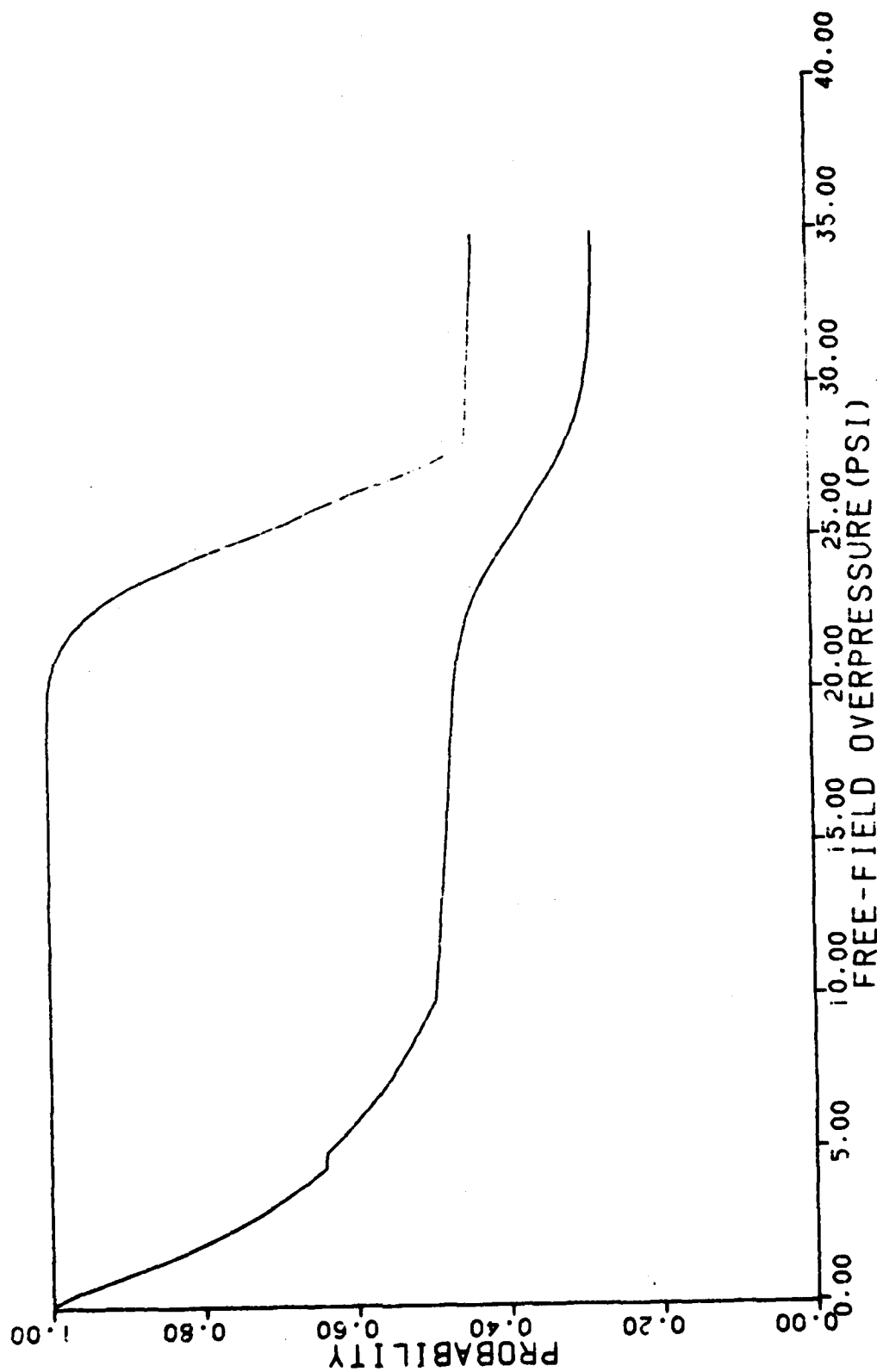


Figure C-78. Probability of people survival (upper and lower bounds) case 8D.

CASE 8E2

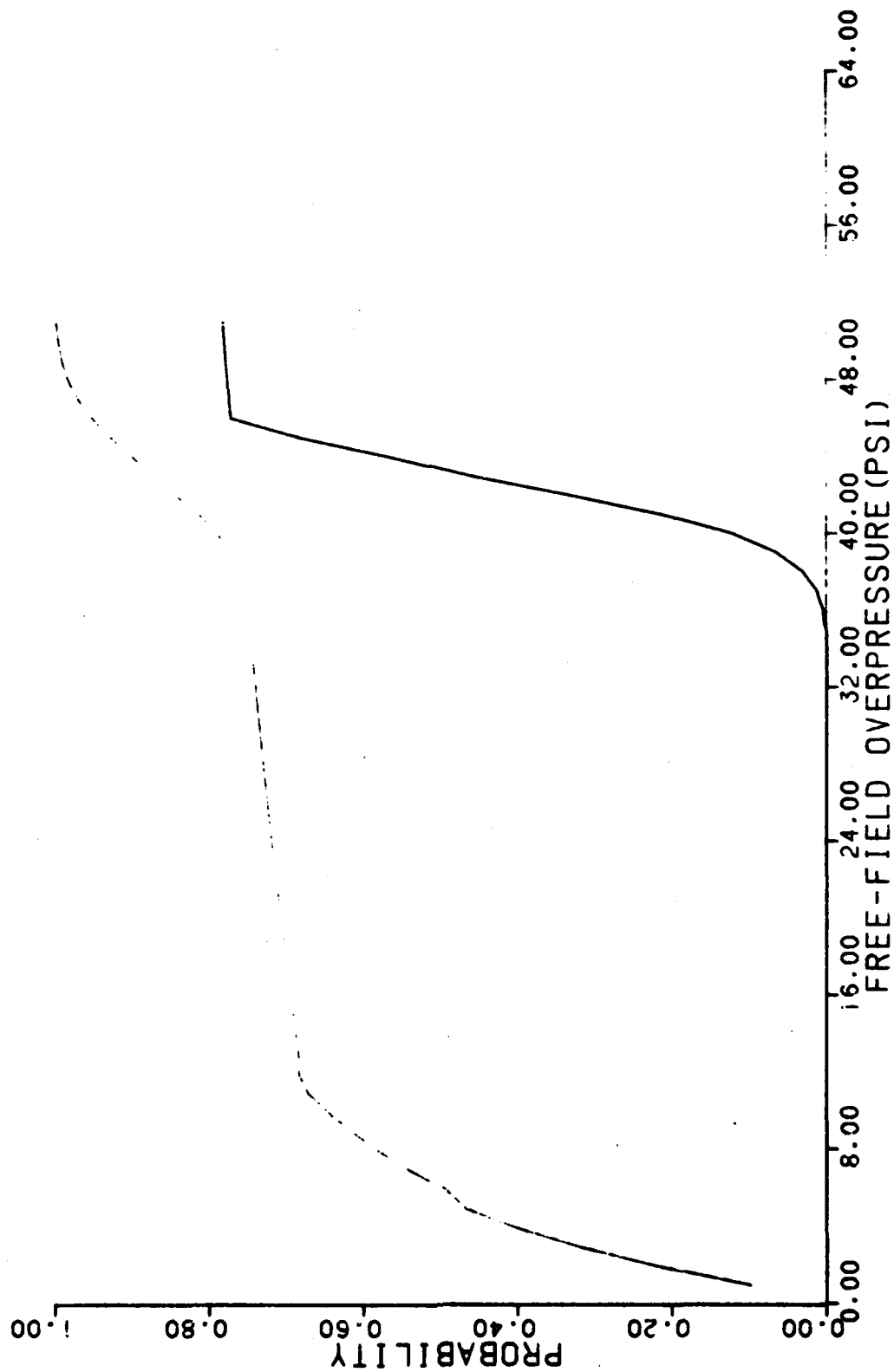


Figure C-79. Probability of slab failure (upper and lower bounds) case 8E.

CASE 8E3

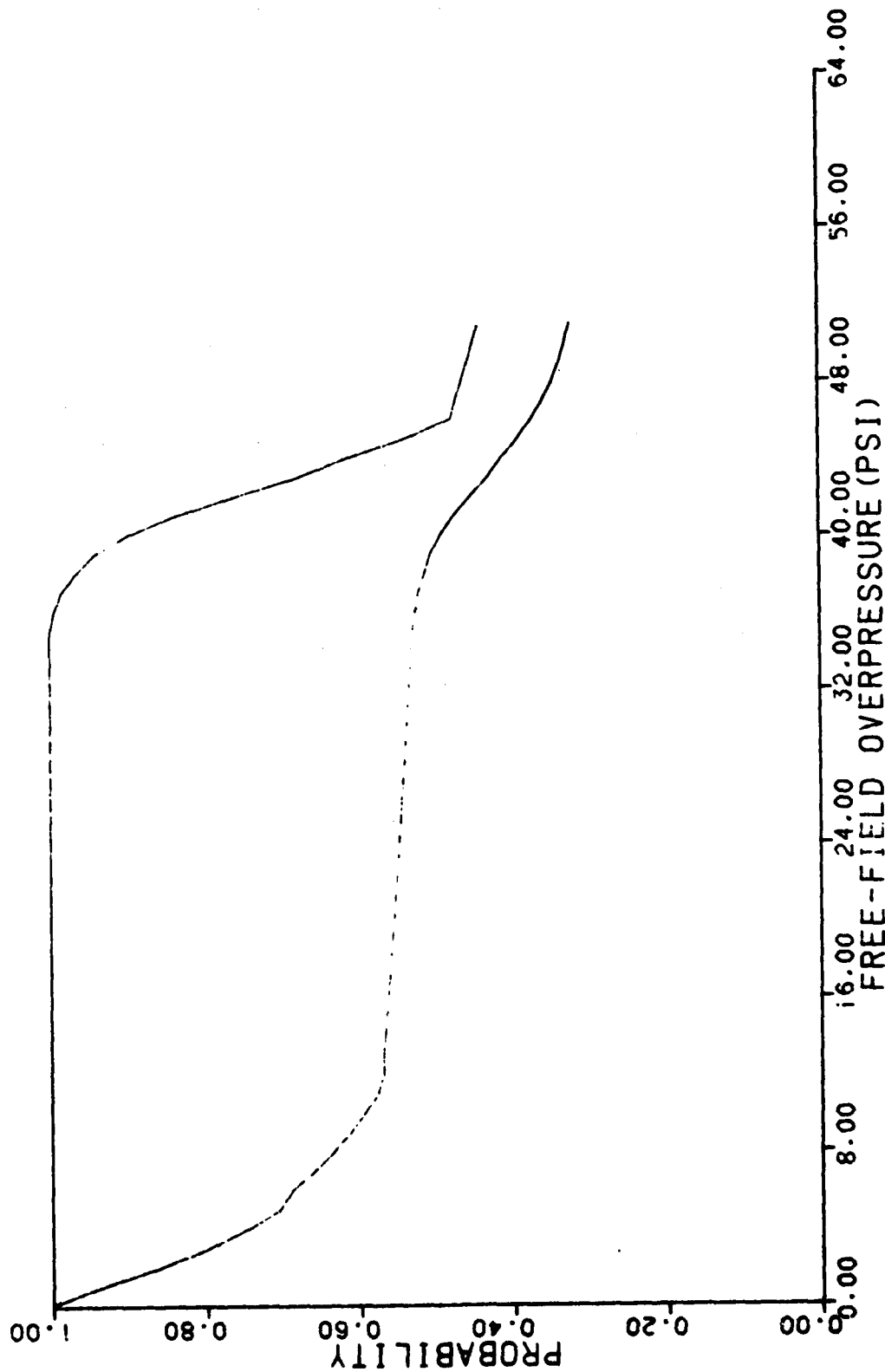


Figure C-80. Probability of people survival (upper and lower bounds) case 8E.

CASE 9A2

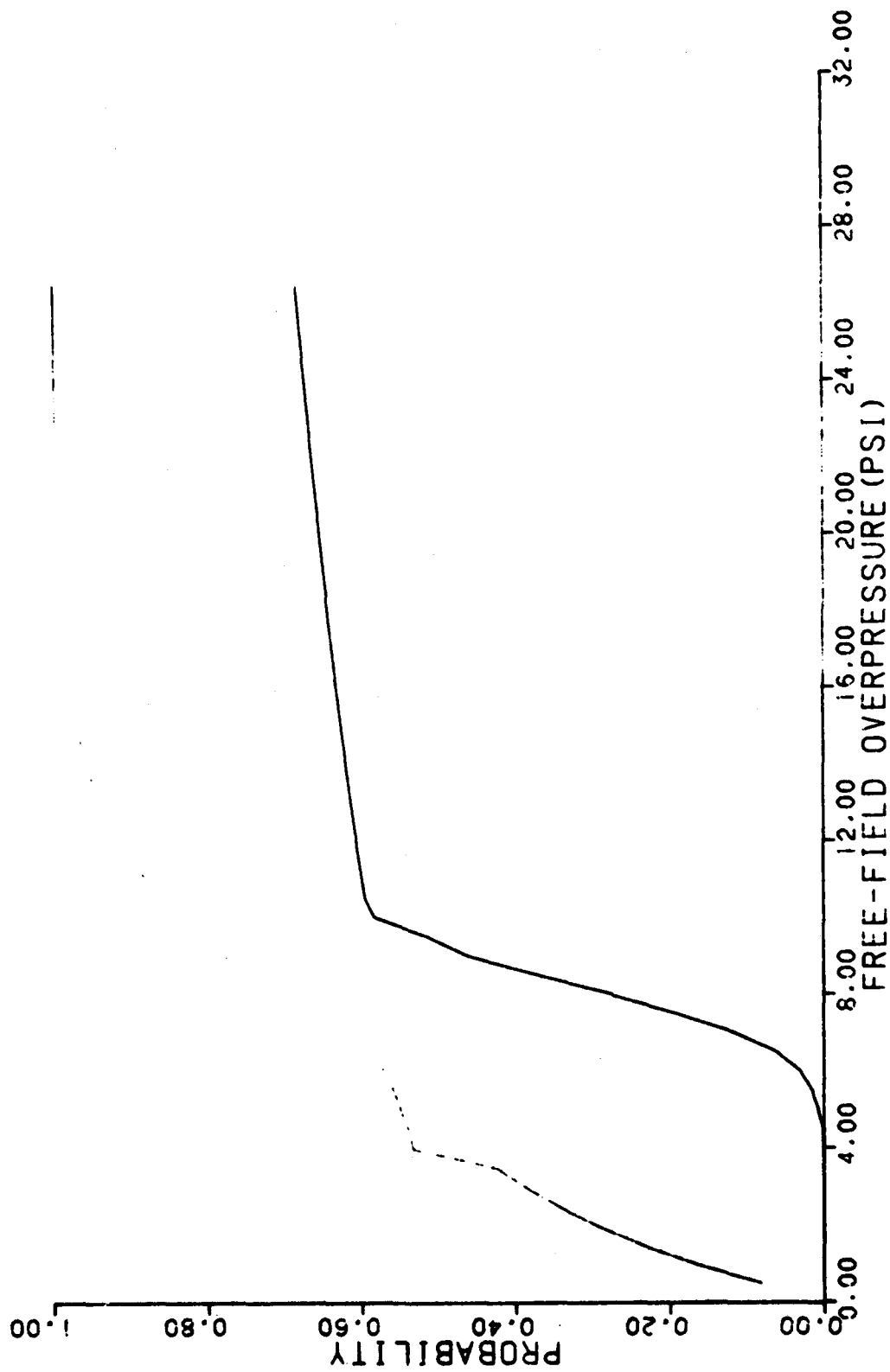


Figure C-81. Probability of slab failure (upper and lower bounds) case 9A.

CASE 9A3

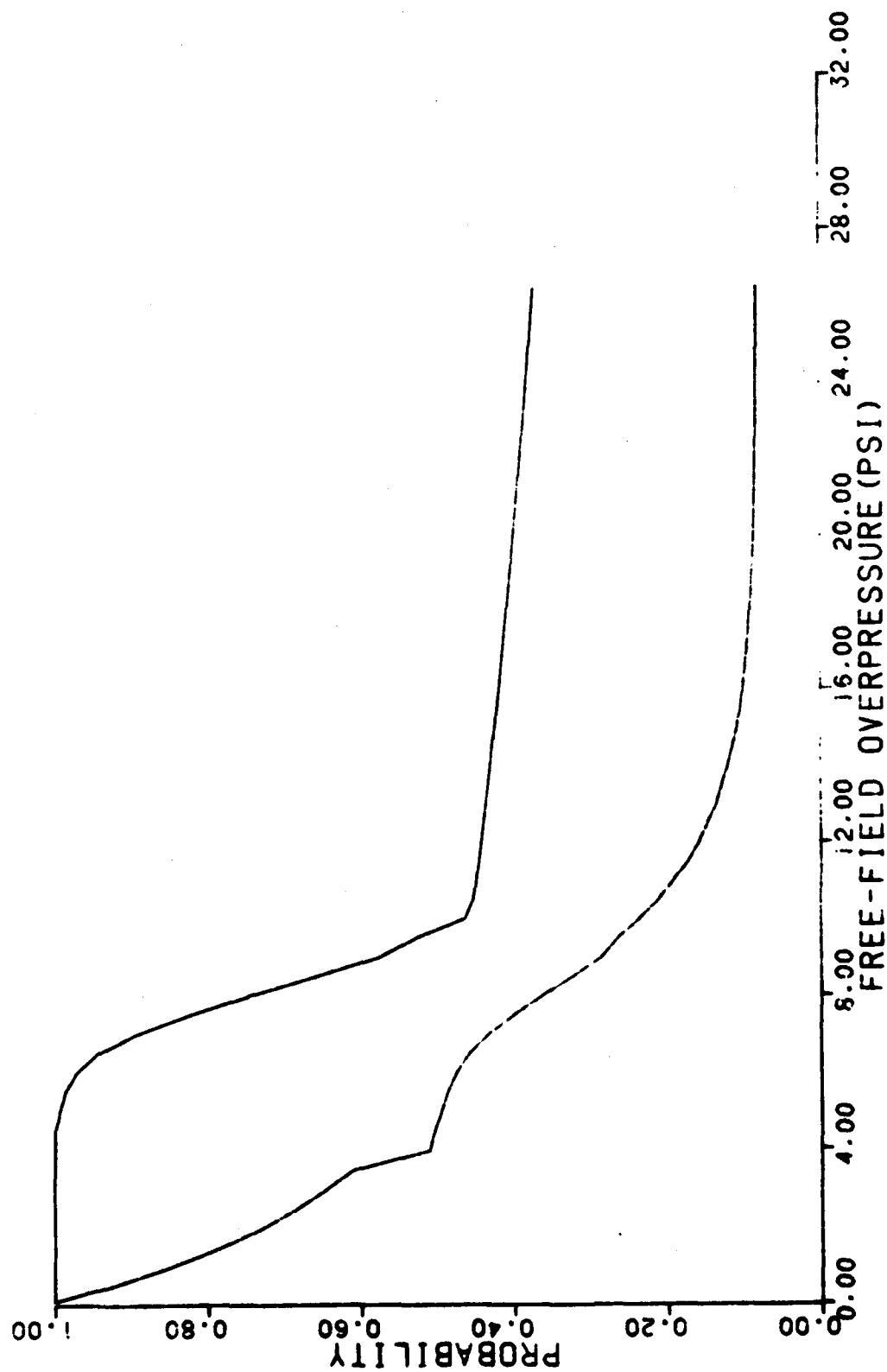


Figure C-82. Probability of people survival (upper and lower bounds) case 9A.

CASE 9B2

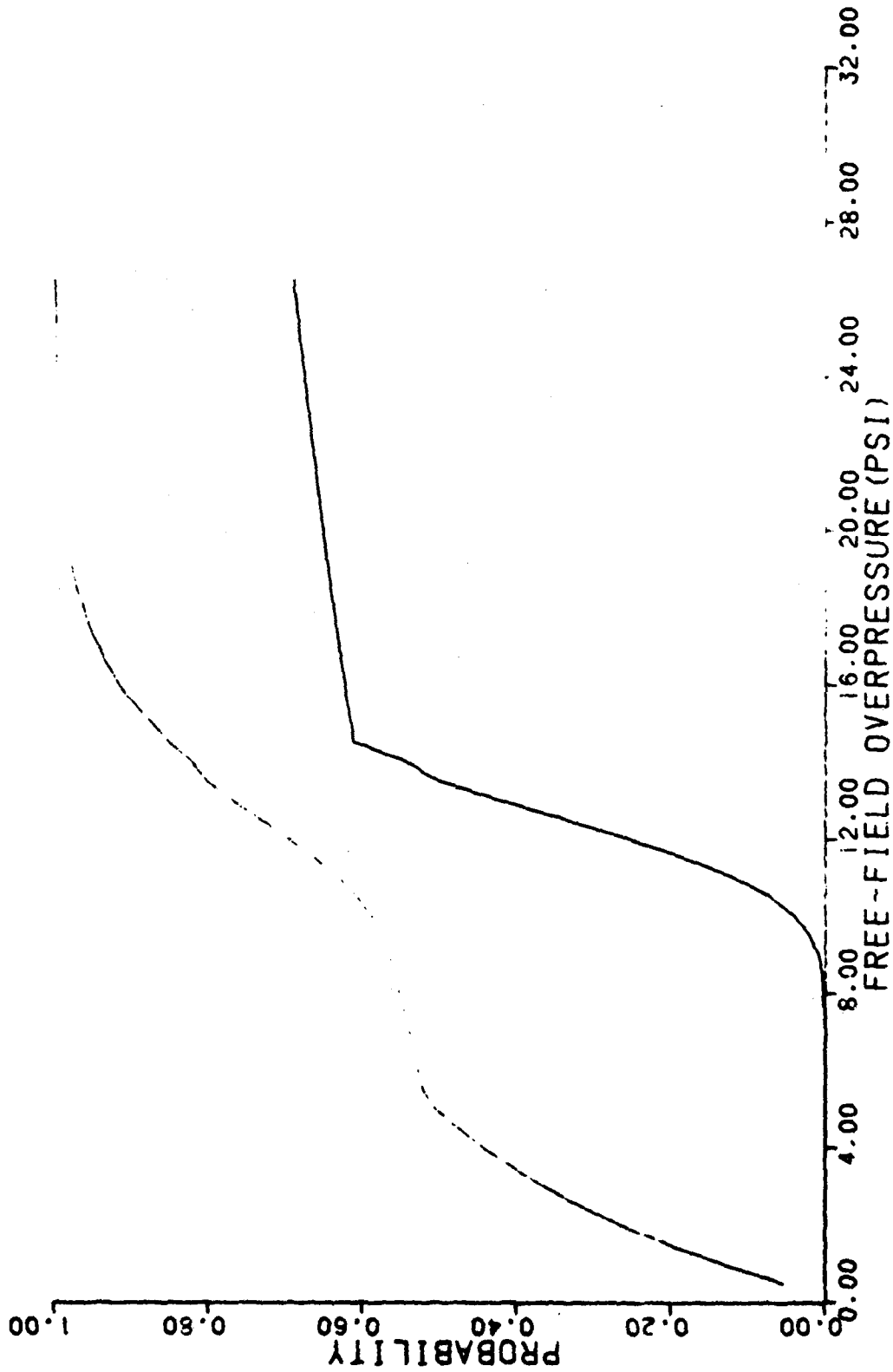


Figure C-83. Probability of slab failure (upper and lower bounds) case 9B.

CASE 9B3

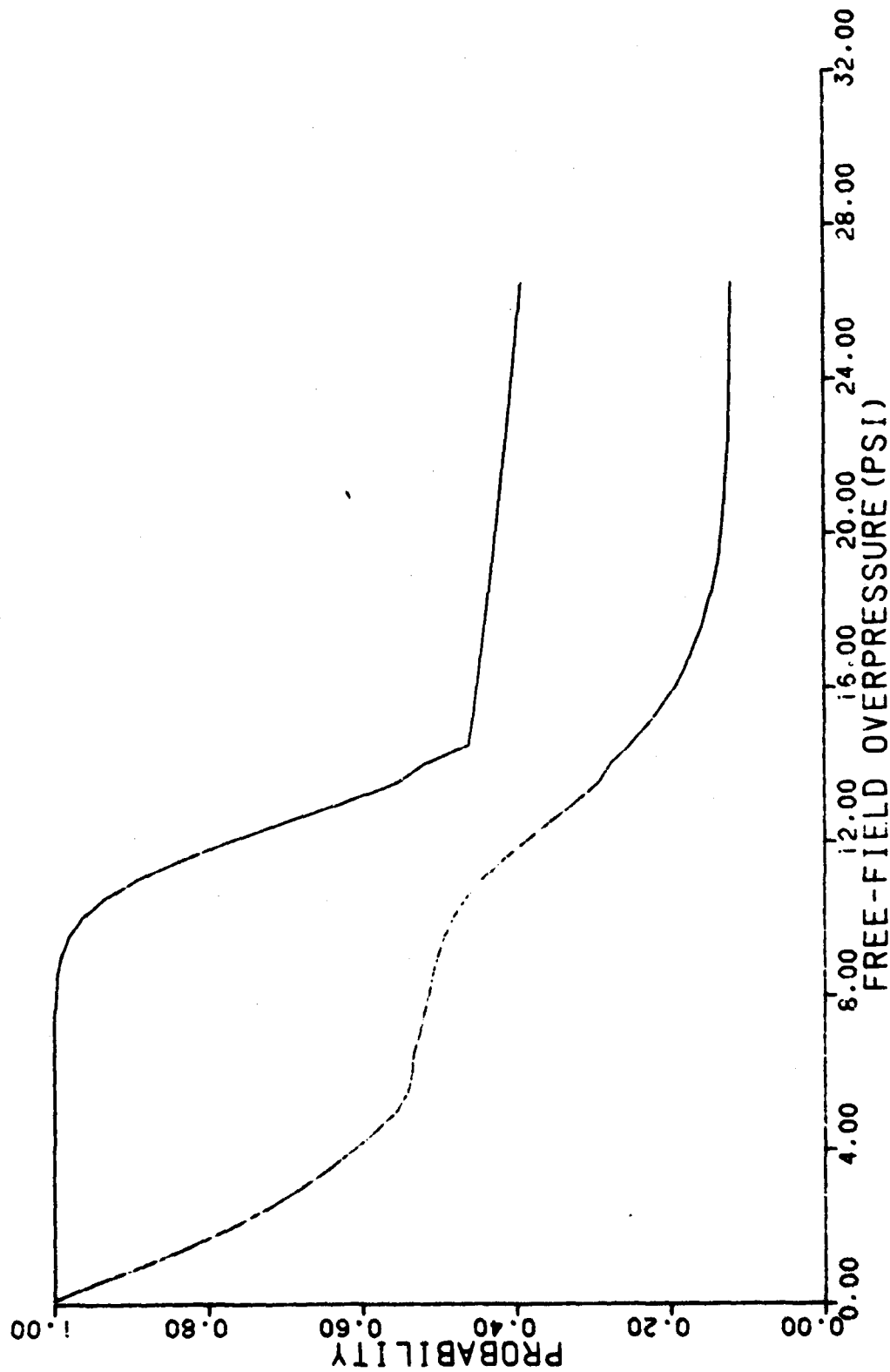


Figure C-84. Probability of people survival (upper and lower bounds) case 9B.

CASE 9C2

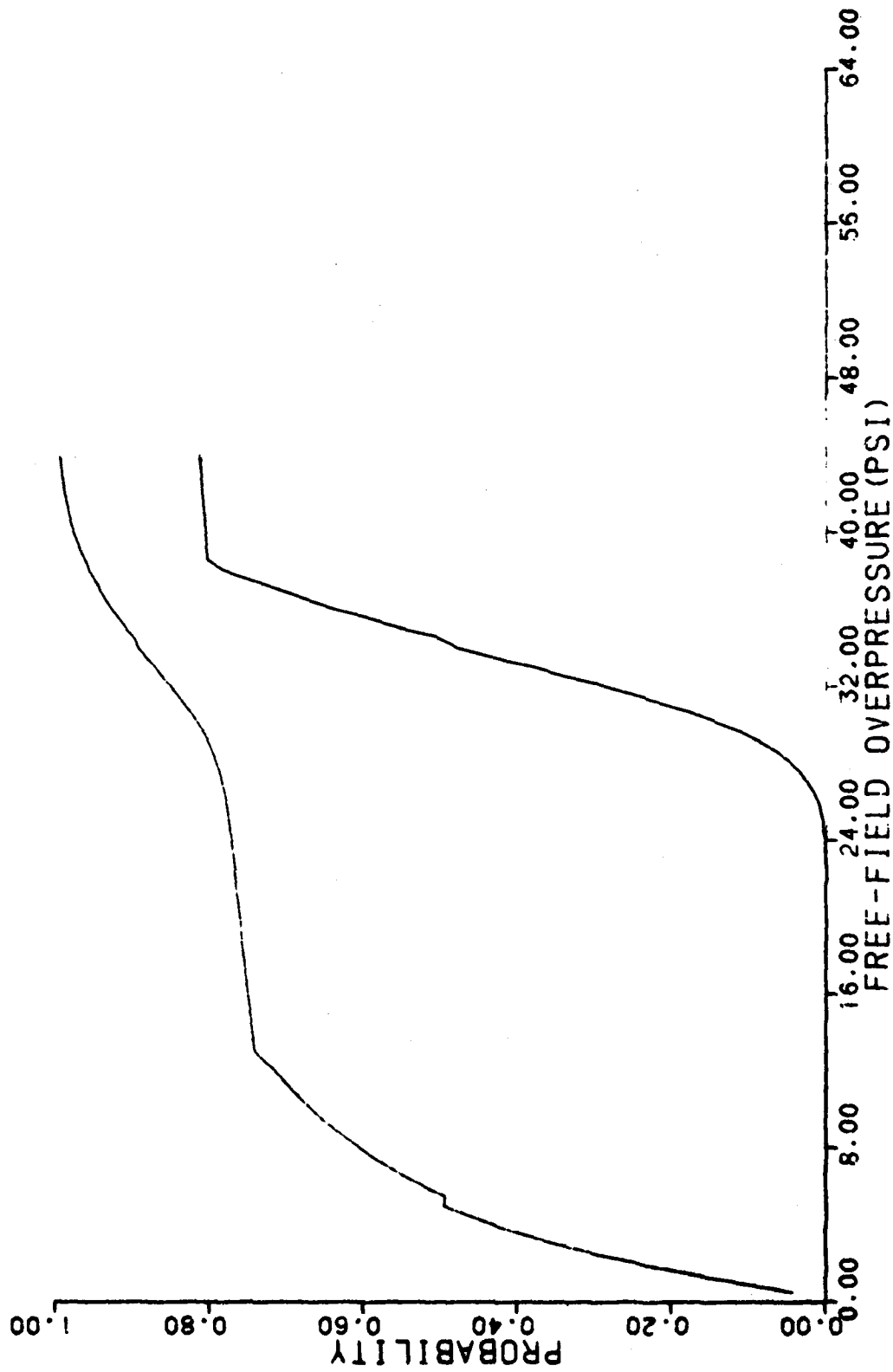


Figure C-85. Probability of slab failure (upper and lower bounds) case 9C.

CASE 9C3

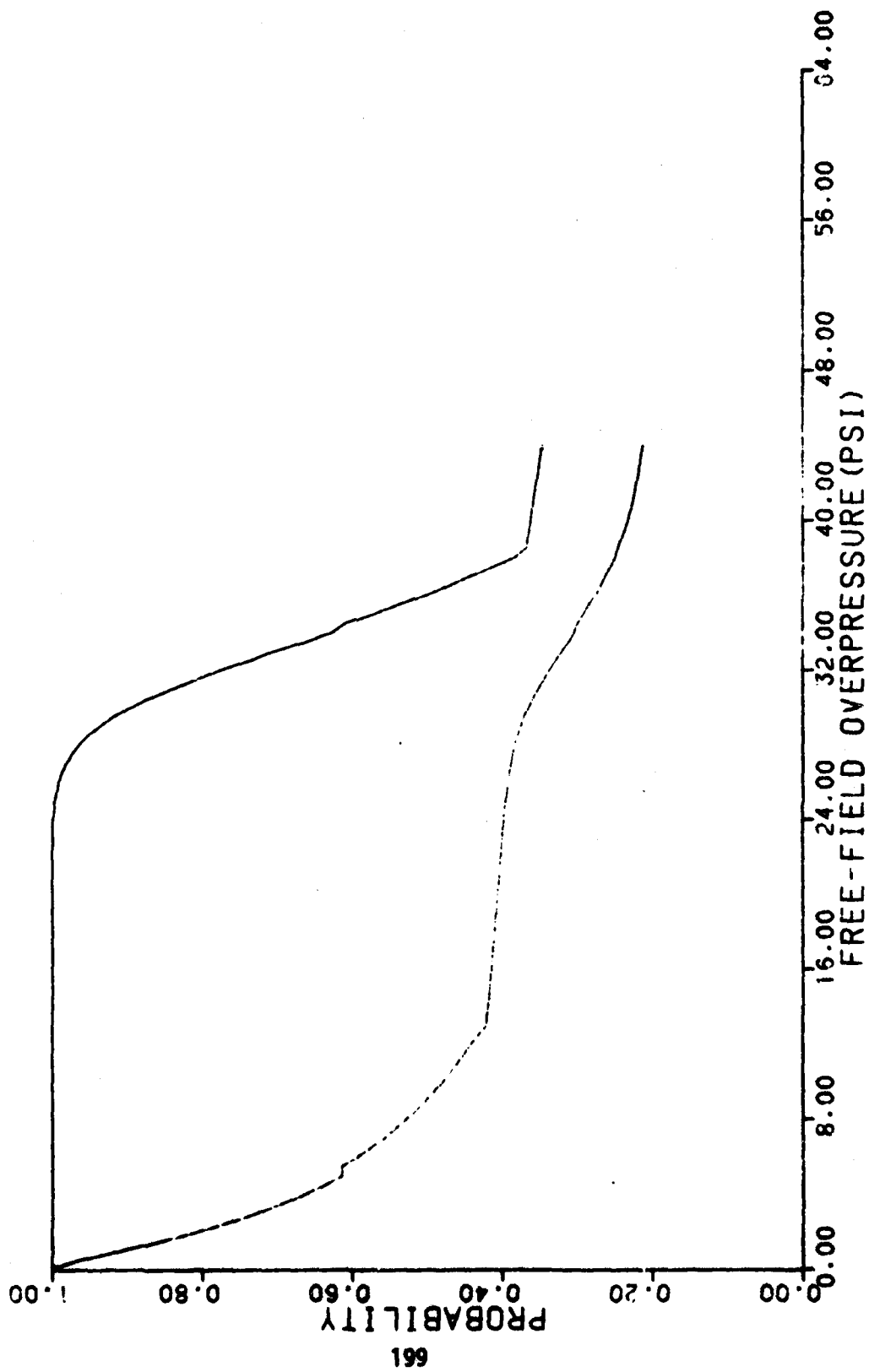


Figure C-86. Probability of people survival (upper and lower bounds) case 9C.

CASE 9D2

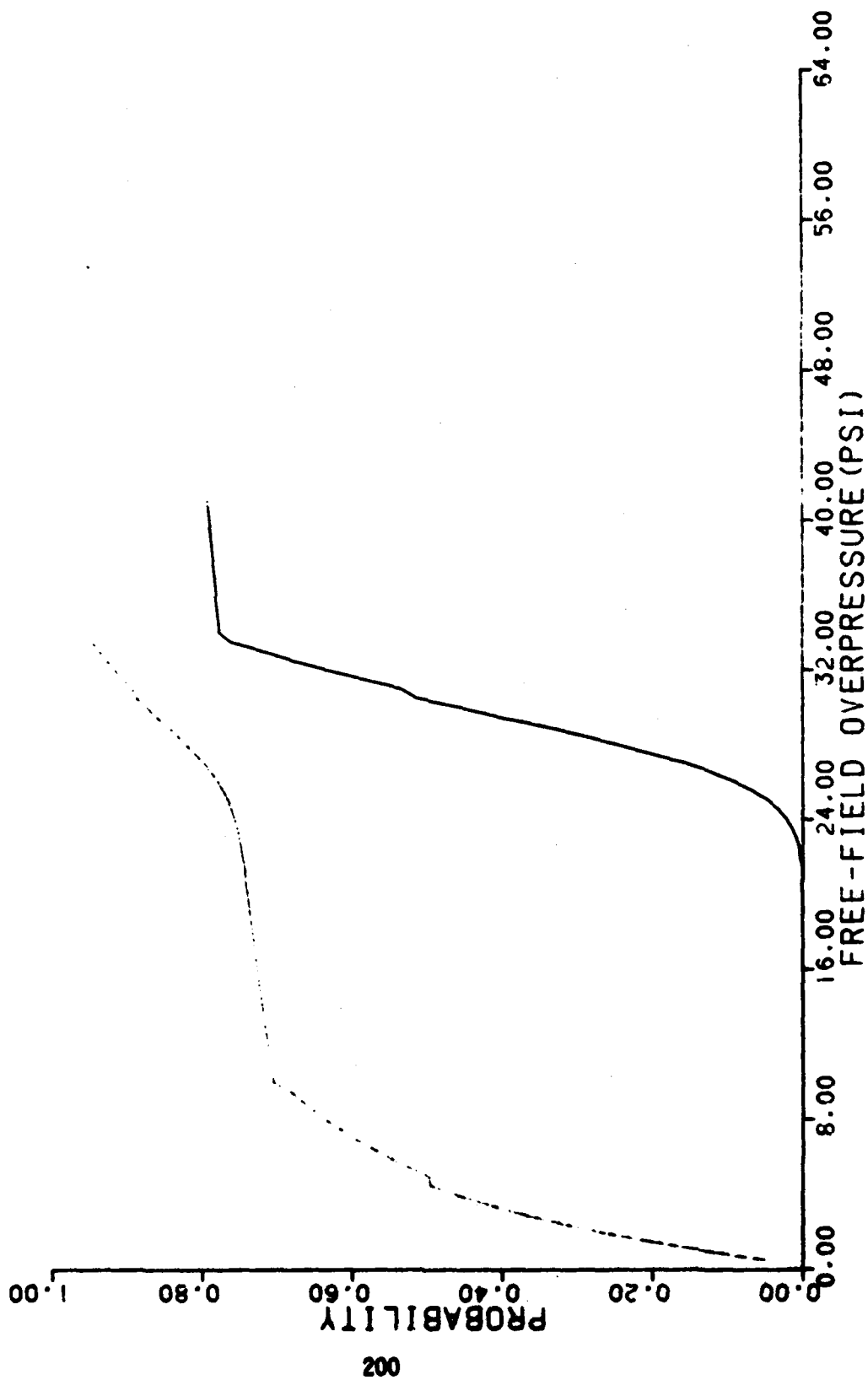


Figure C-87. Probability of slab failure (upper and lower bounds) case 9D.

CASE 9D3

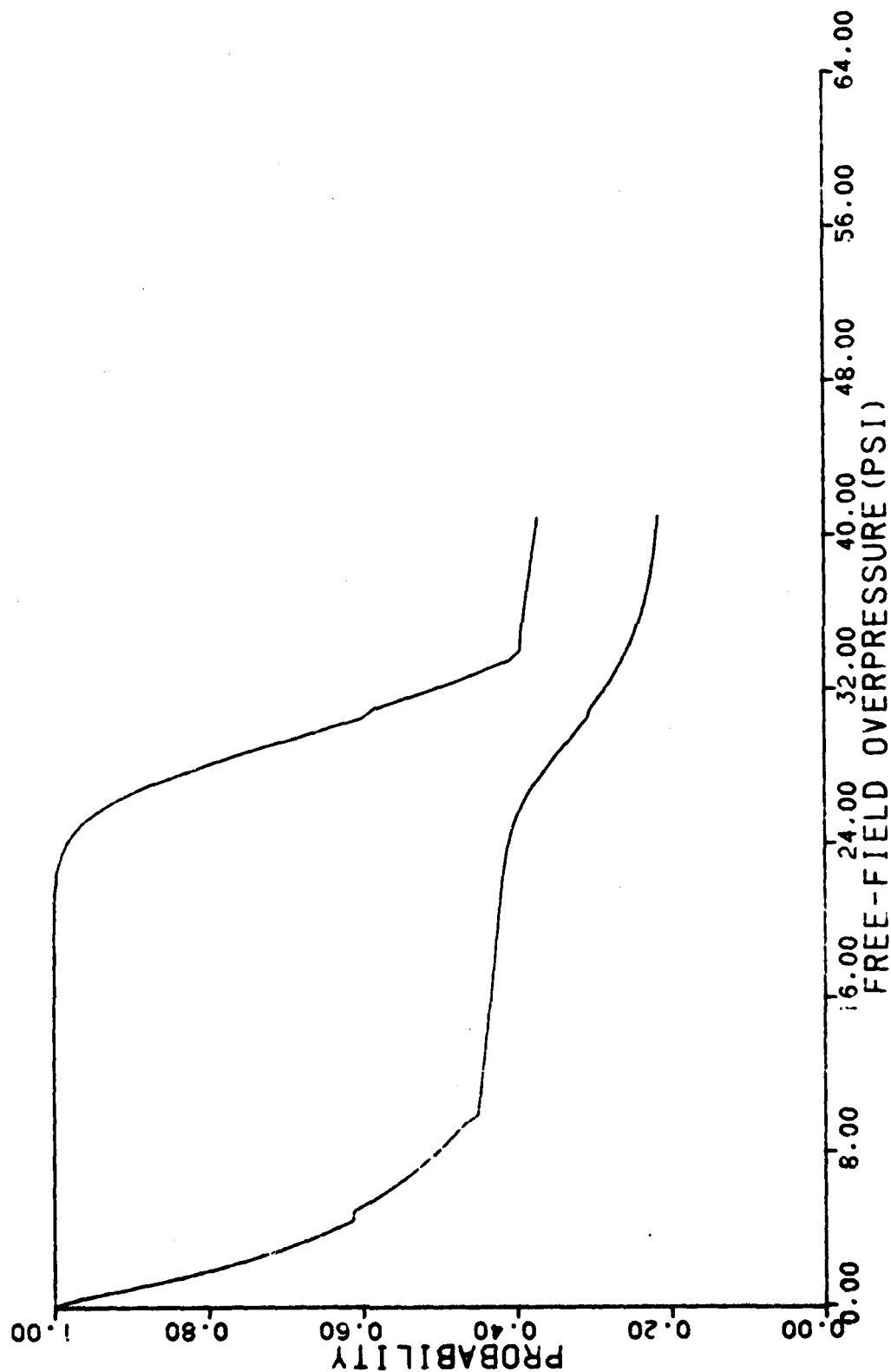


Figure C-88. Probability of people survival (upper and lower bounds) case 9D.

CASE 9E2

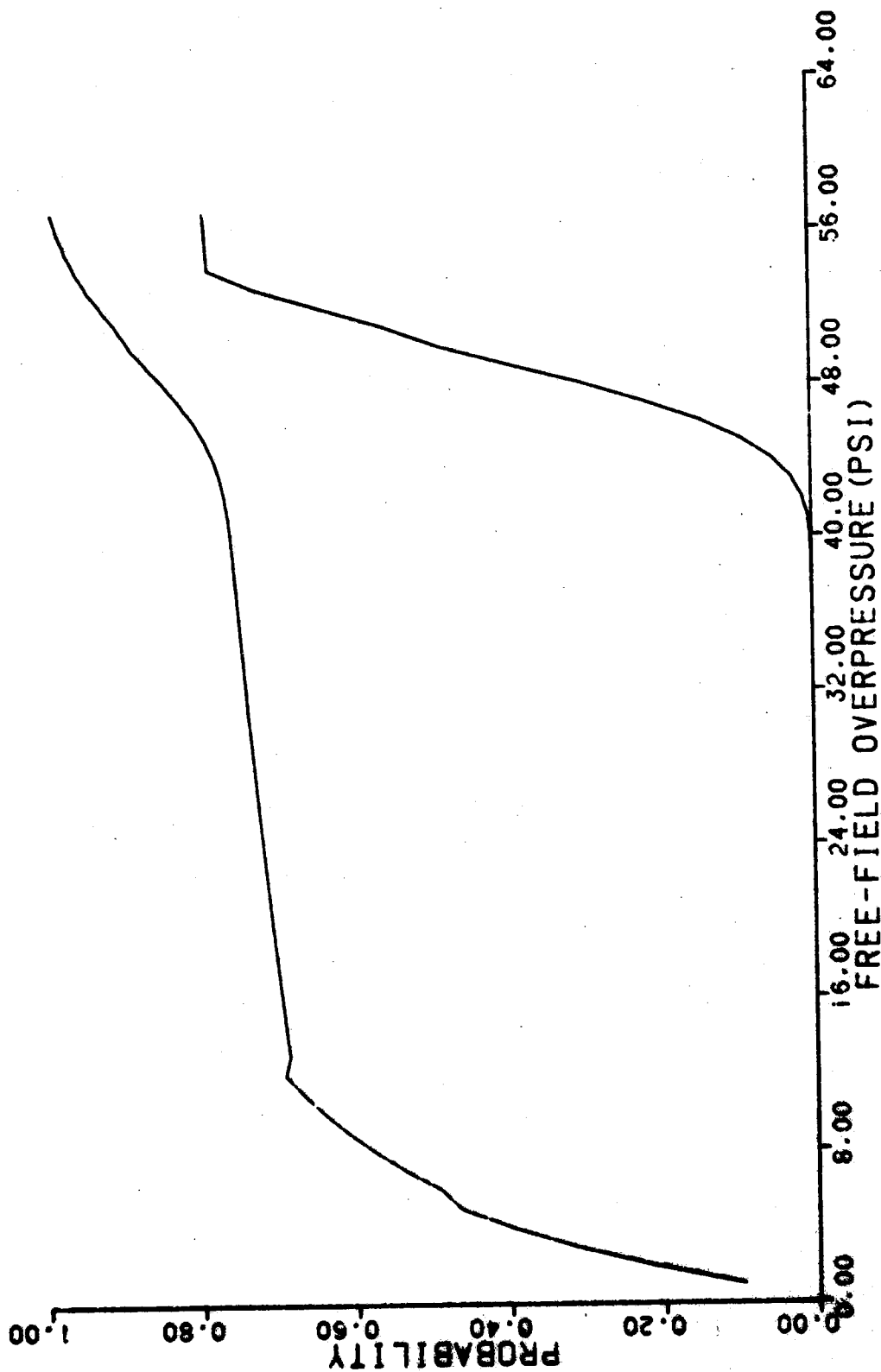


Figure C-89. Probability of slab failure (upper and lower bounds) case 9E.

CASE 9E3

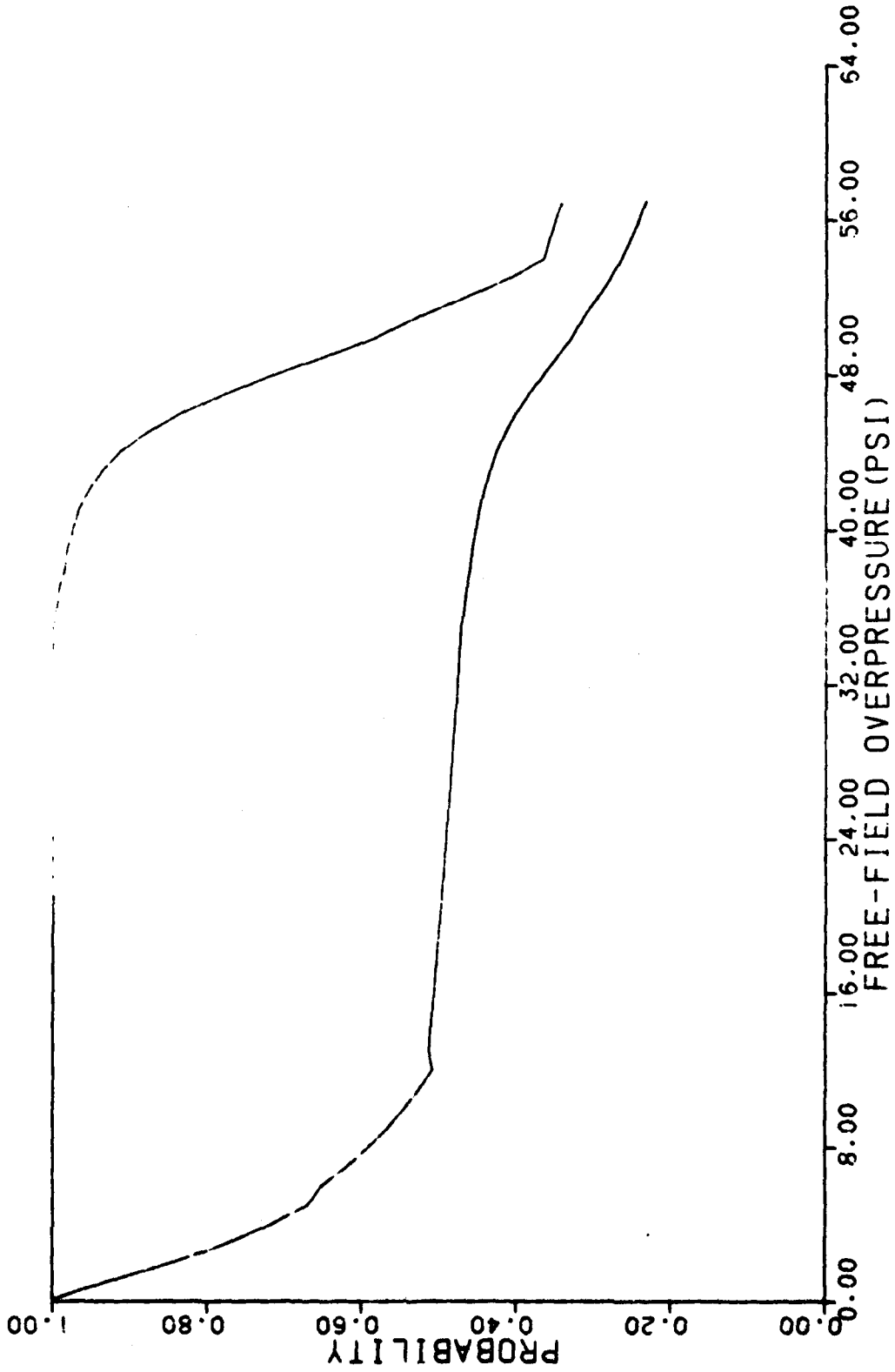


Figure C-90. Probability of people survival (upper and lower bounds) case 9E.

CASE 10A2

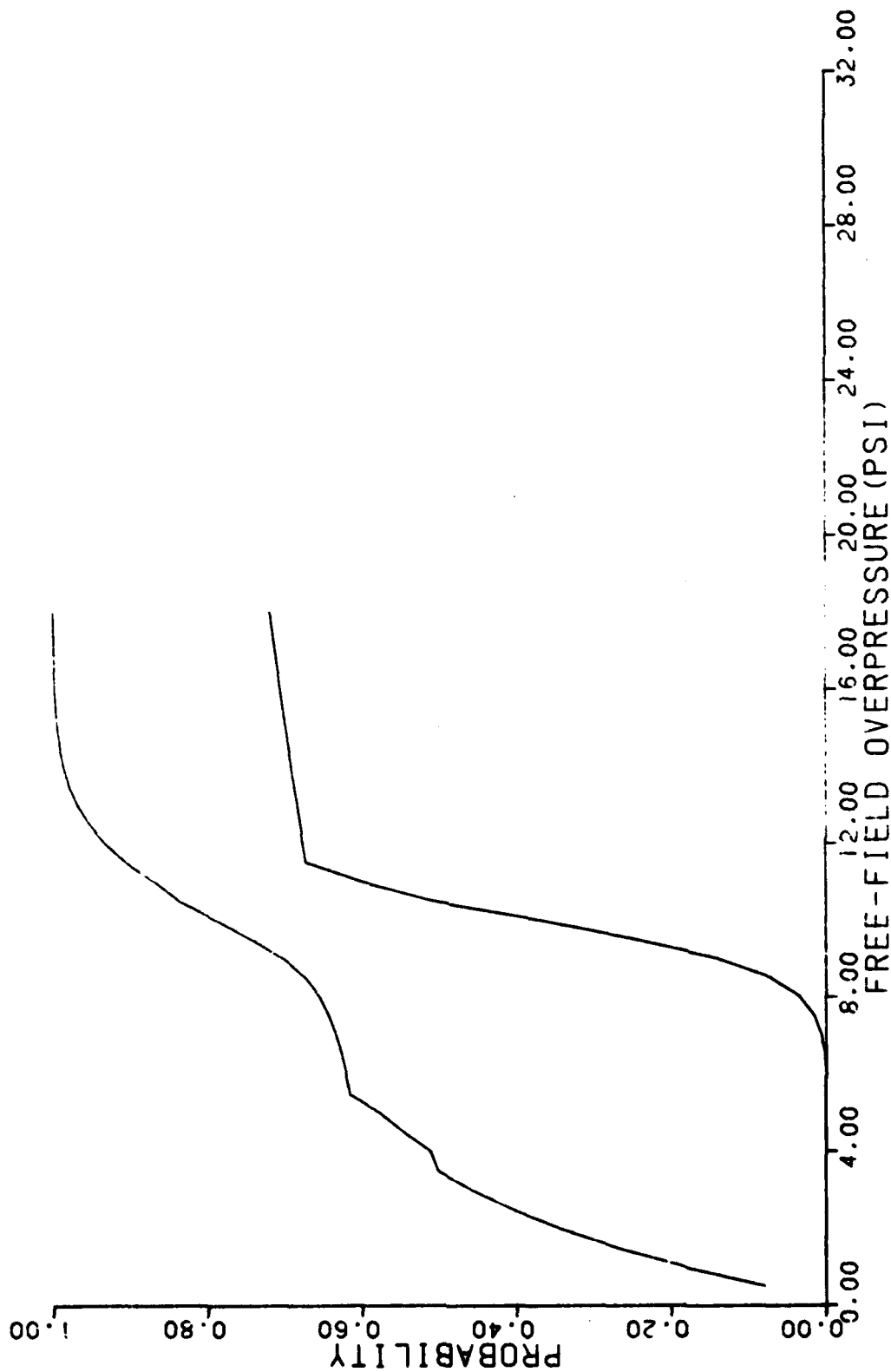


Figure C-91. Probability of slab failure (upper and lower bounds) case 10A.

CASE 10A3

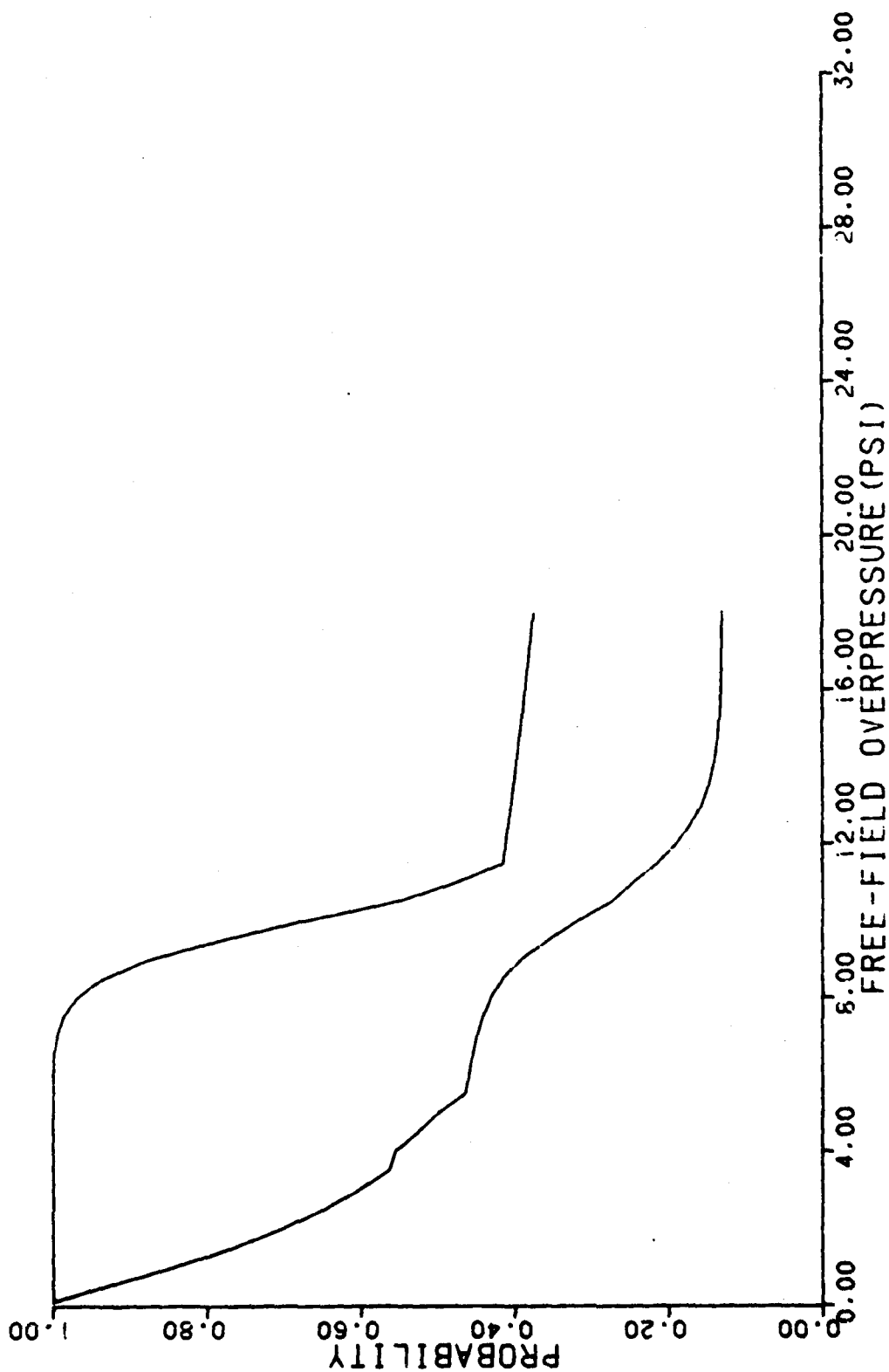


Figure C-92. Probability of people survival (upper and lower bounds) case 10A.

CASE 10B2

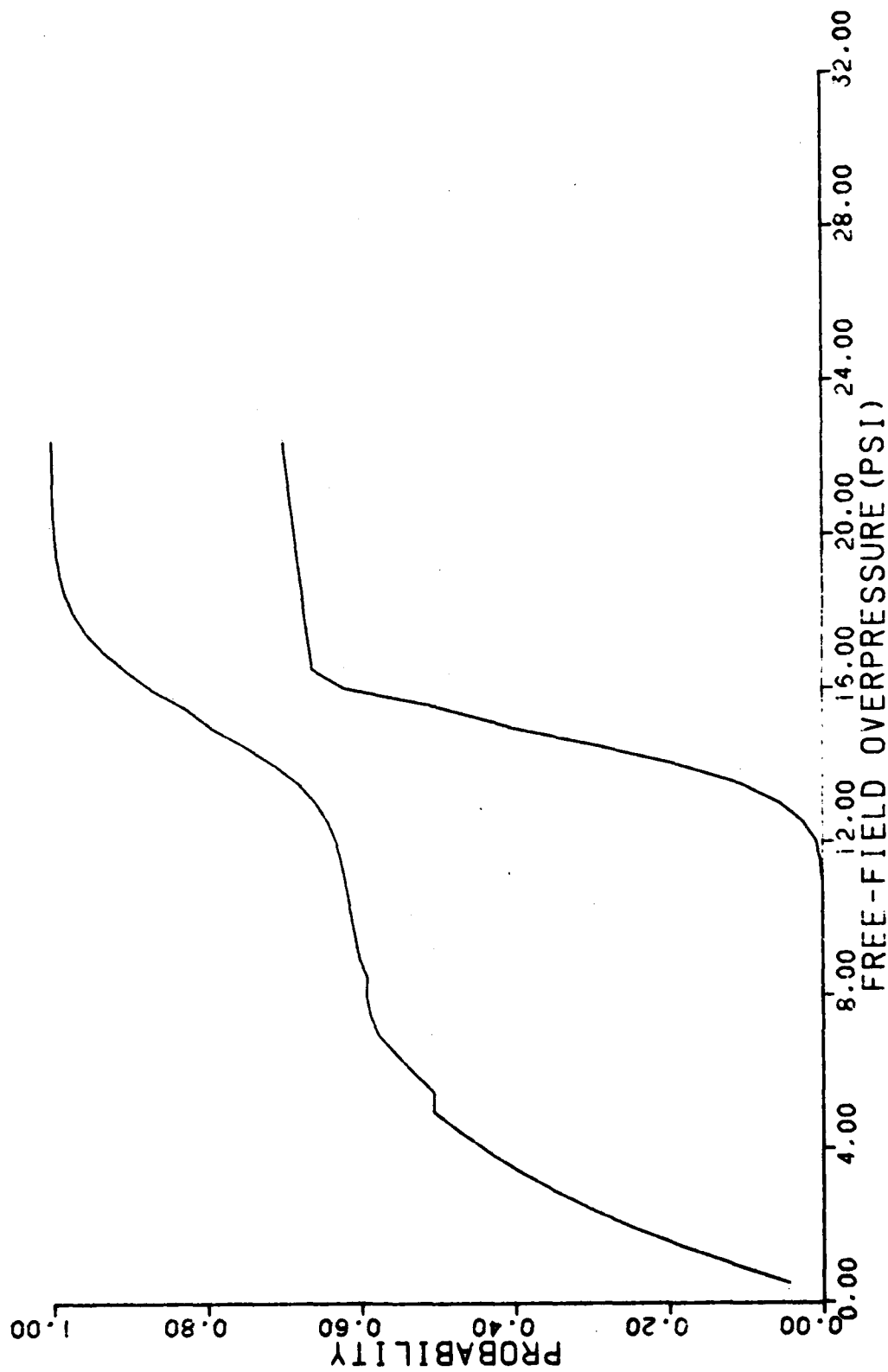


Figure C-93. Probability of slab failure (upper and lower bounds) case 10B.

CASE 10B3

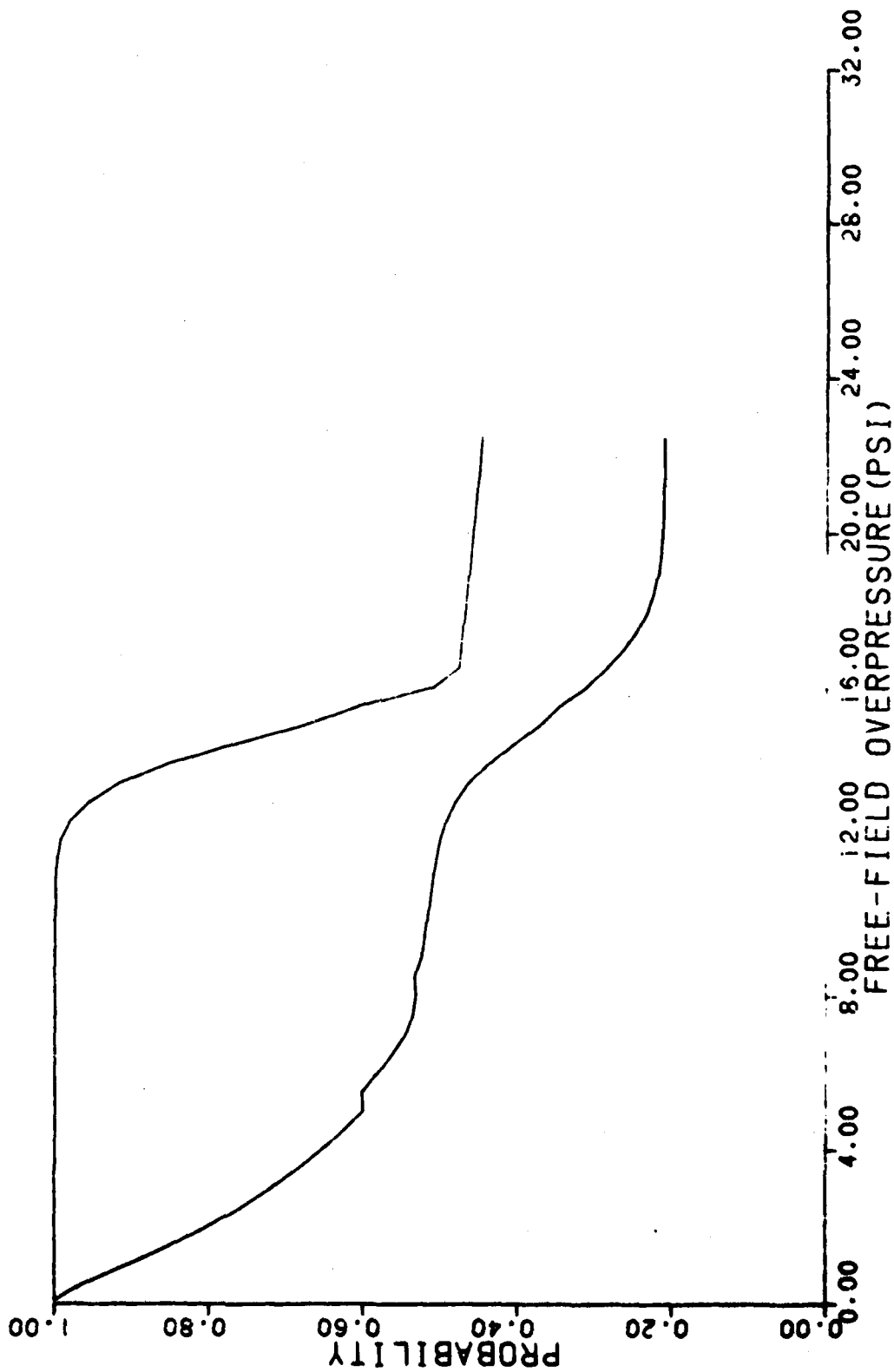


Figure C-94. Probability of people survival (upper and lower bounds) case 10B.

CASE 10C2

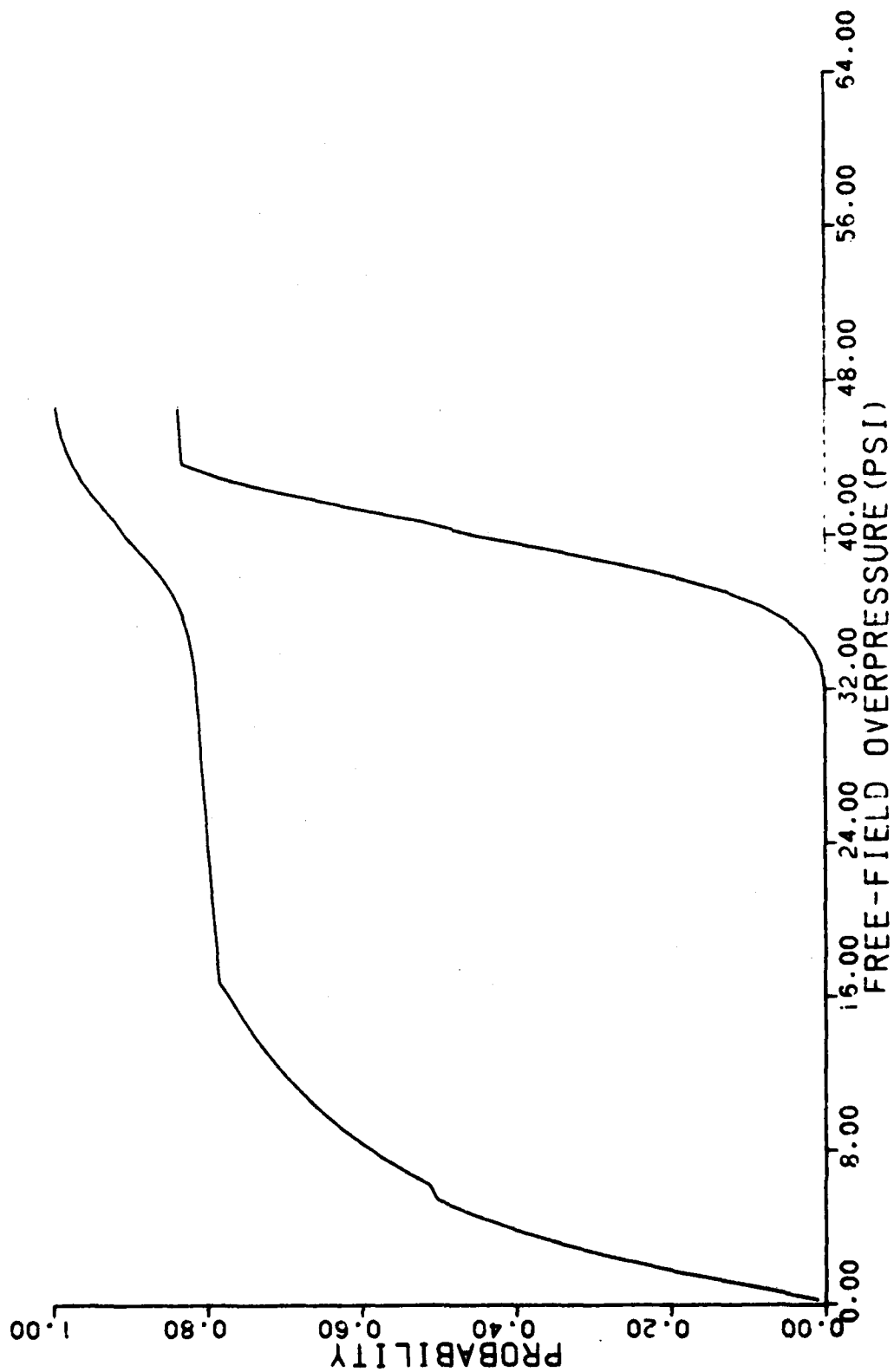


Figure C-95. Probability of slab failure (upper and lower bounds) case 10C.

CASE 10C3

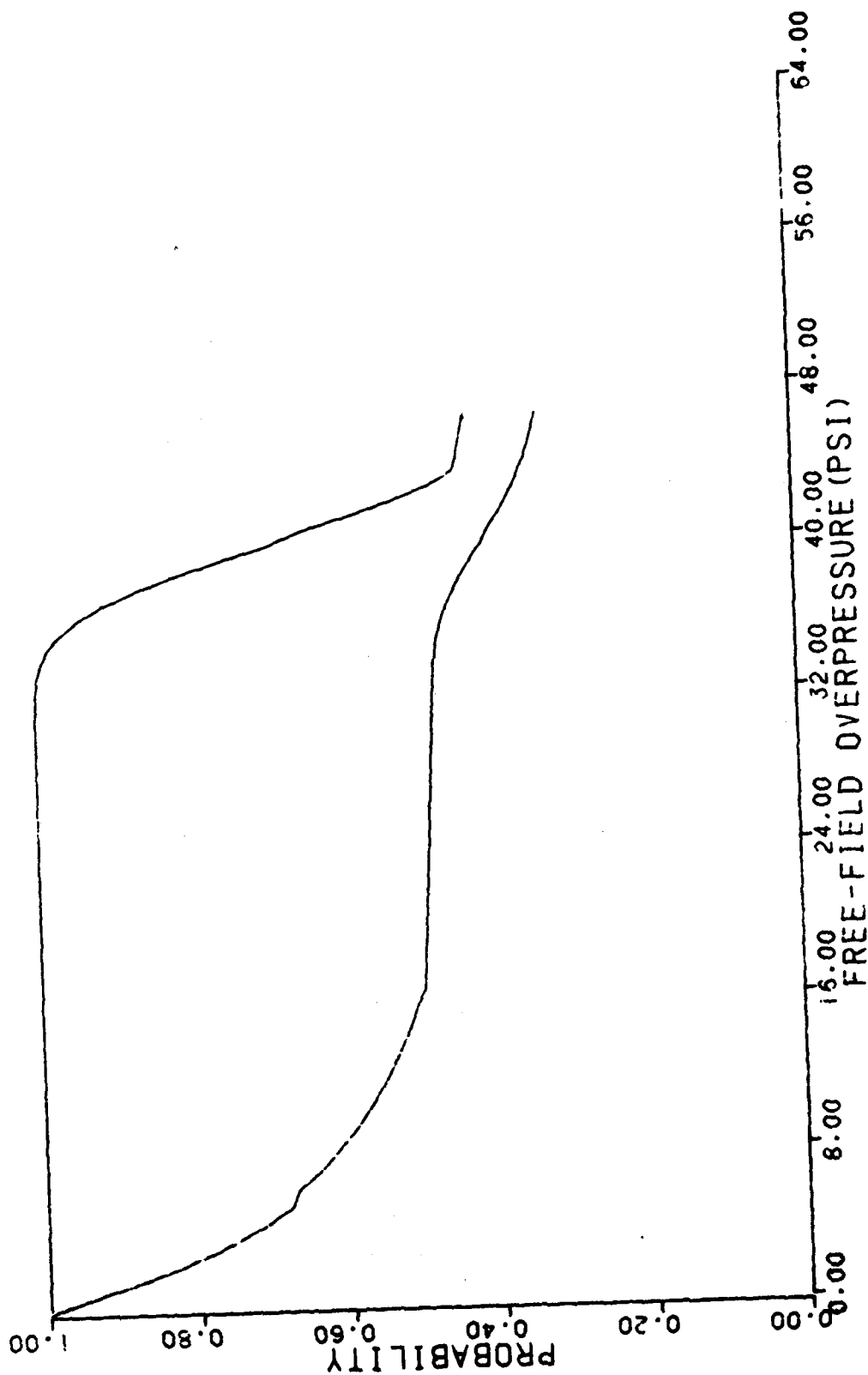


Figure C-96. Probability of people survival (upper and lower bounds) case 10C.

CASE 10D2

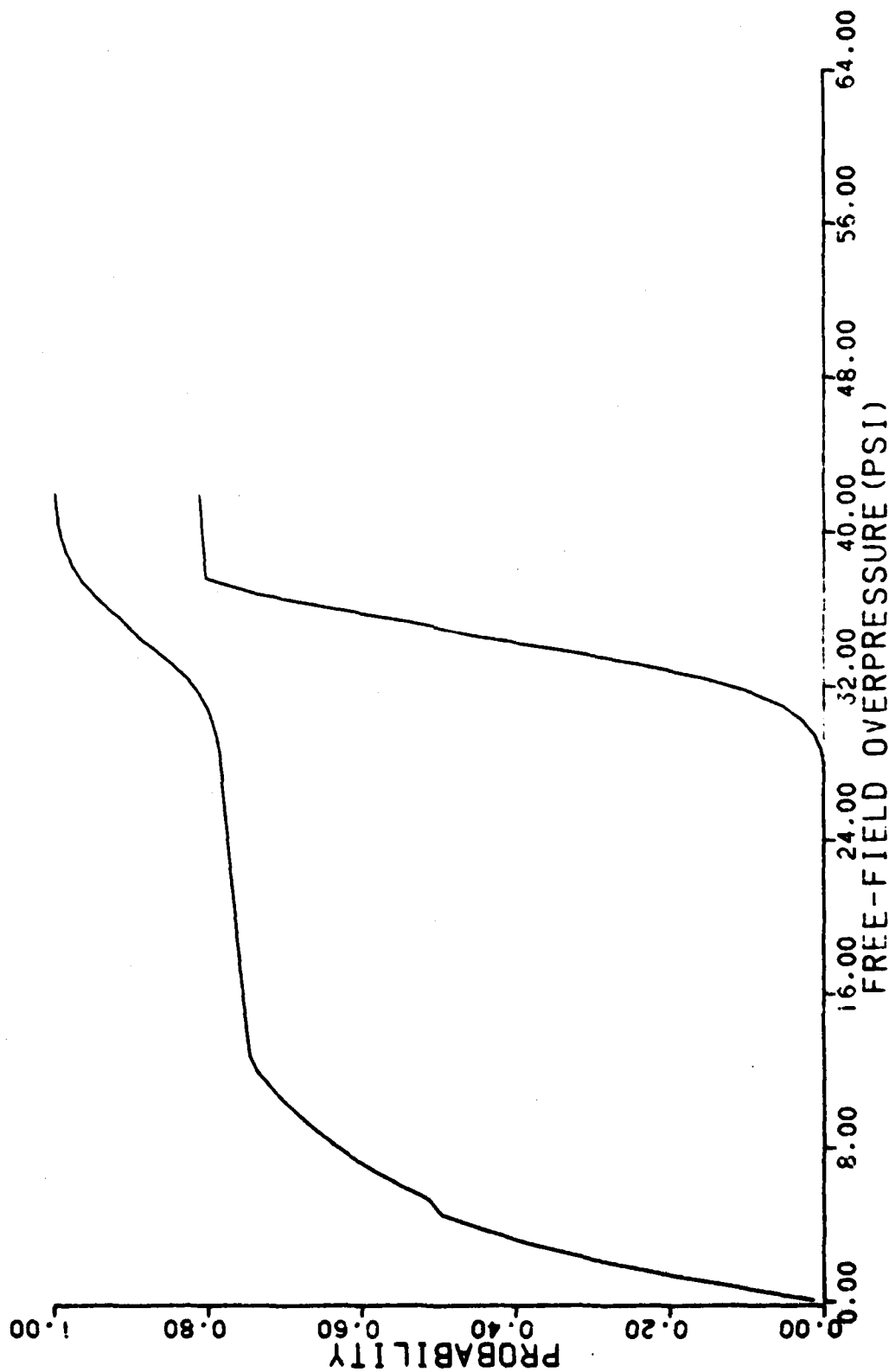


Figure C-97. Probability of slab failure (upper and lower bounds) case 10D.

CASE 10D3

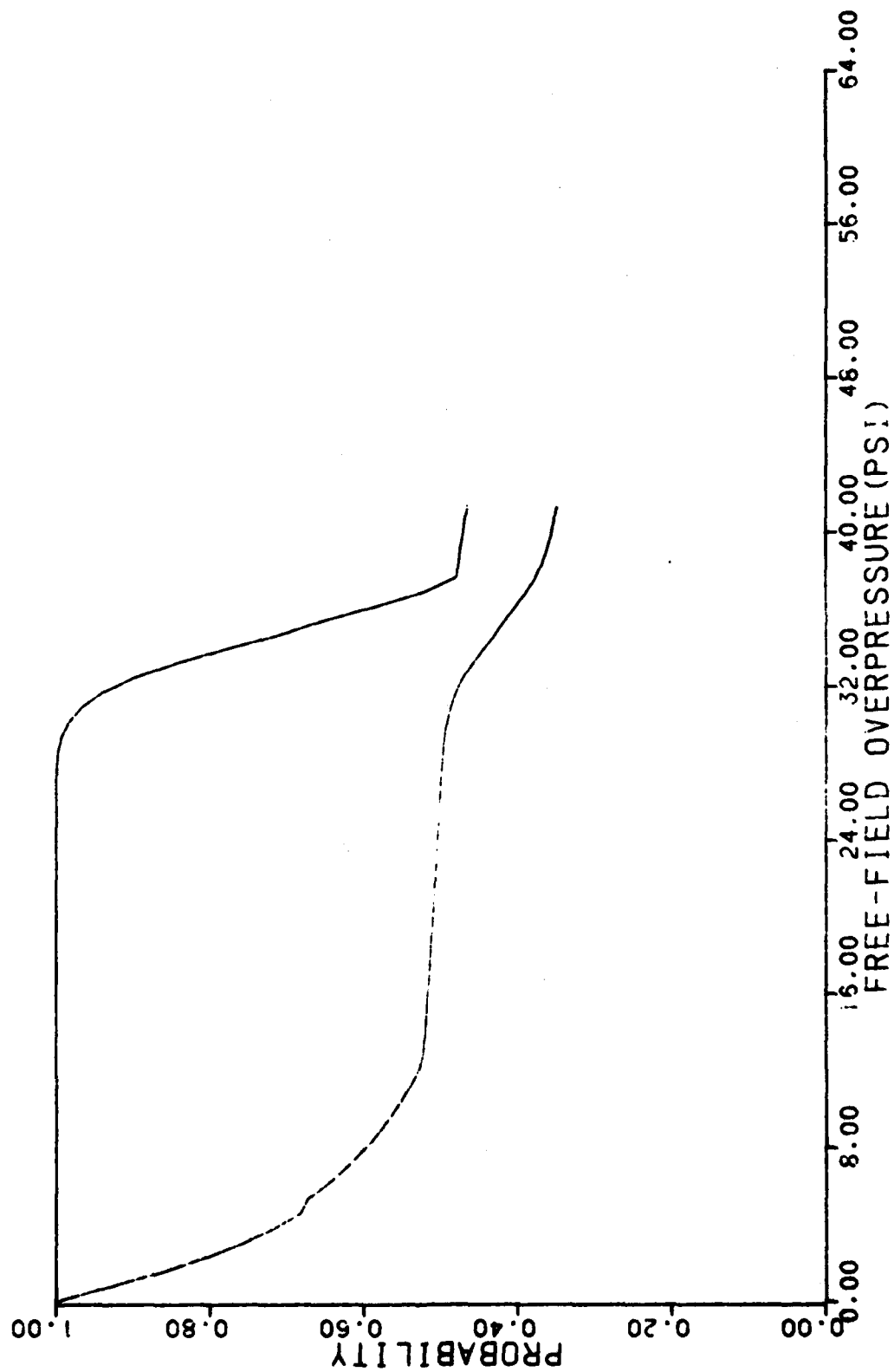


Figure C-98. Probability of people survival (upper and lower bounds) case 10D.

CASE 10E2

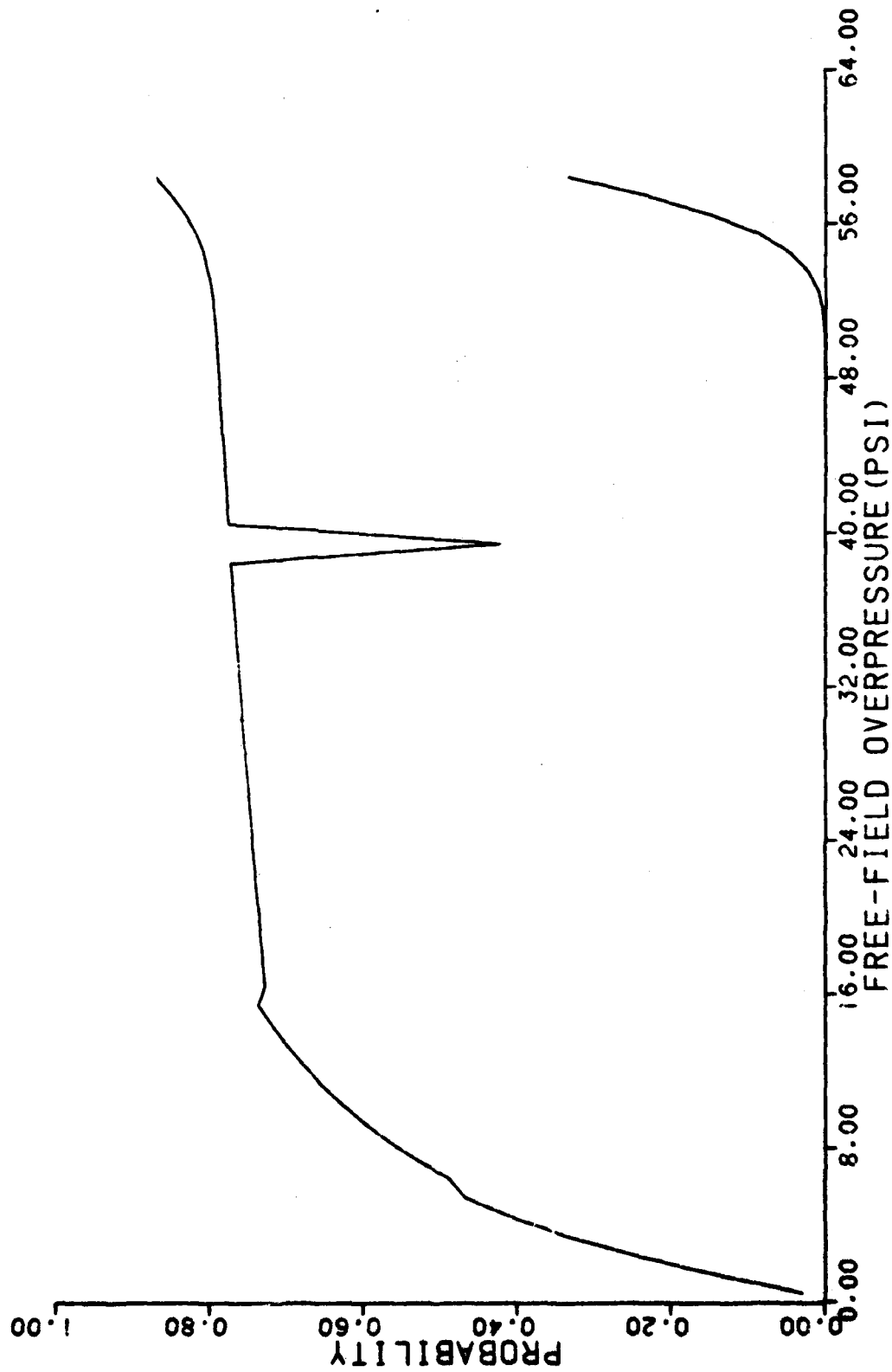


Figure C-99. Probability of slab failure (upper and lower bounds) case 10E.

CASE 10E3

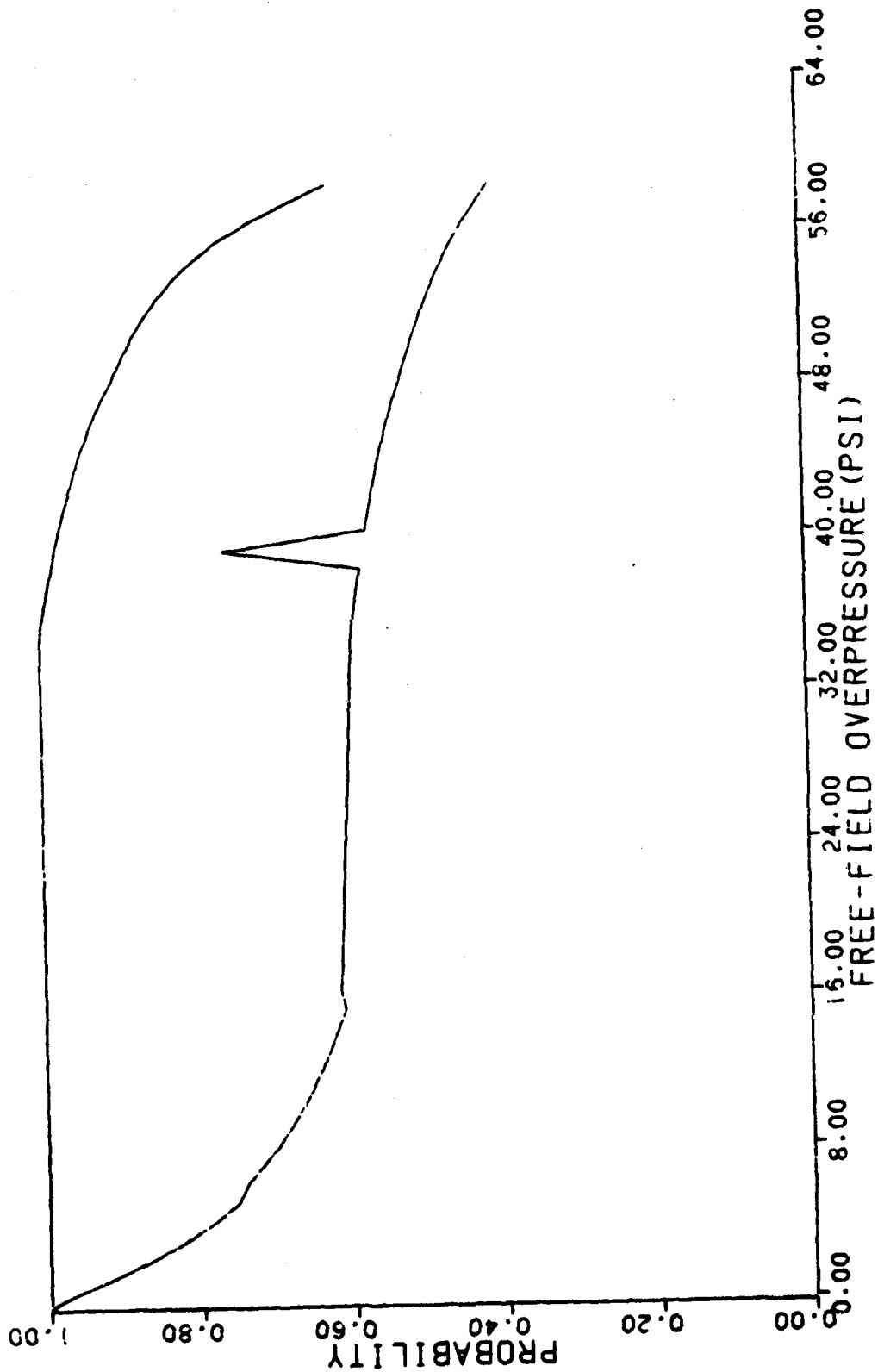


Figure C-100. Probability of people survival (upper and lower bounds) case 10E.

CASE 11A2

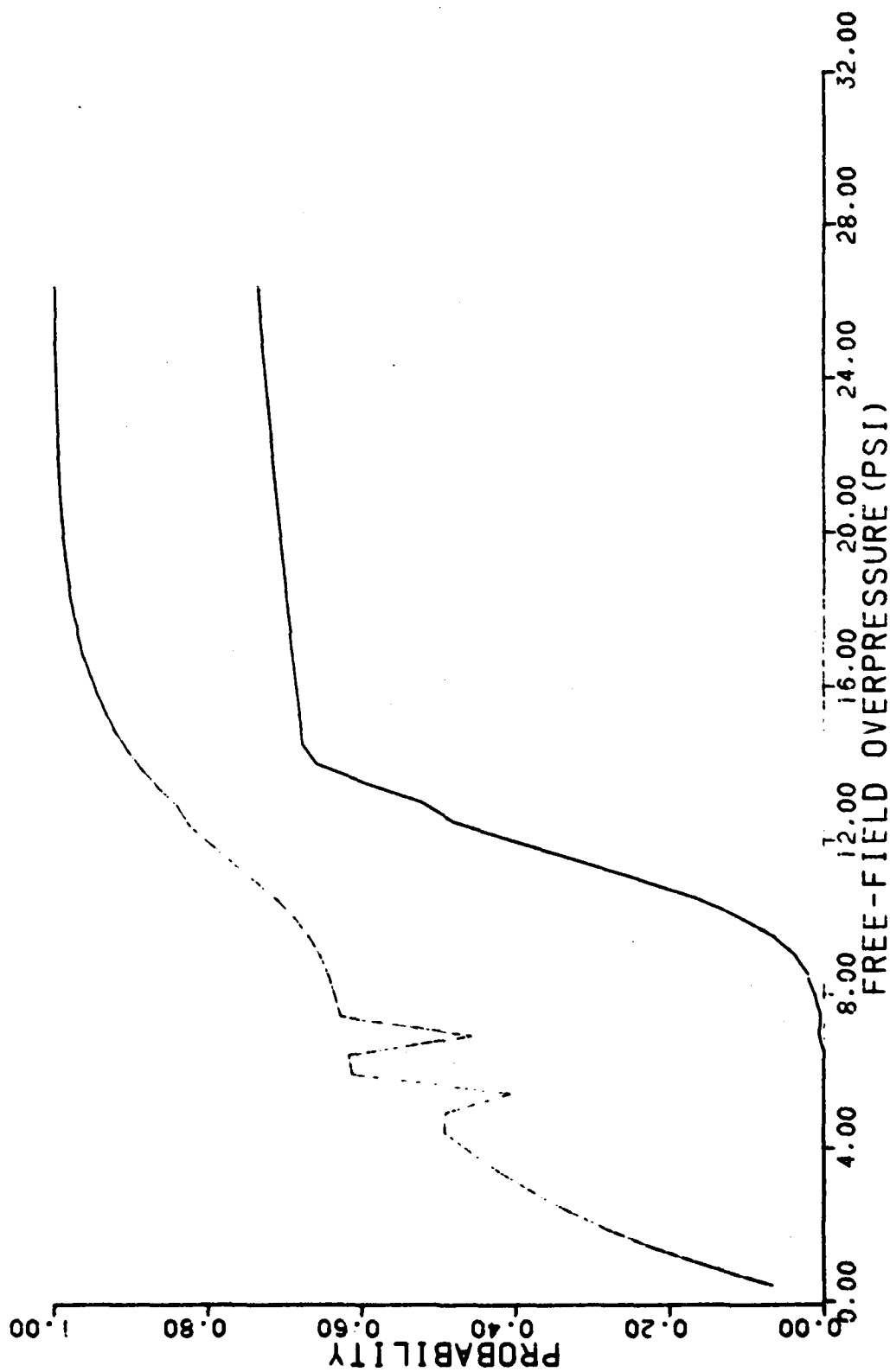


Figure C-101. Probability of slab failure (upper and lower bounds) case 11A.

CASE 11A3

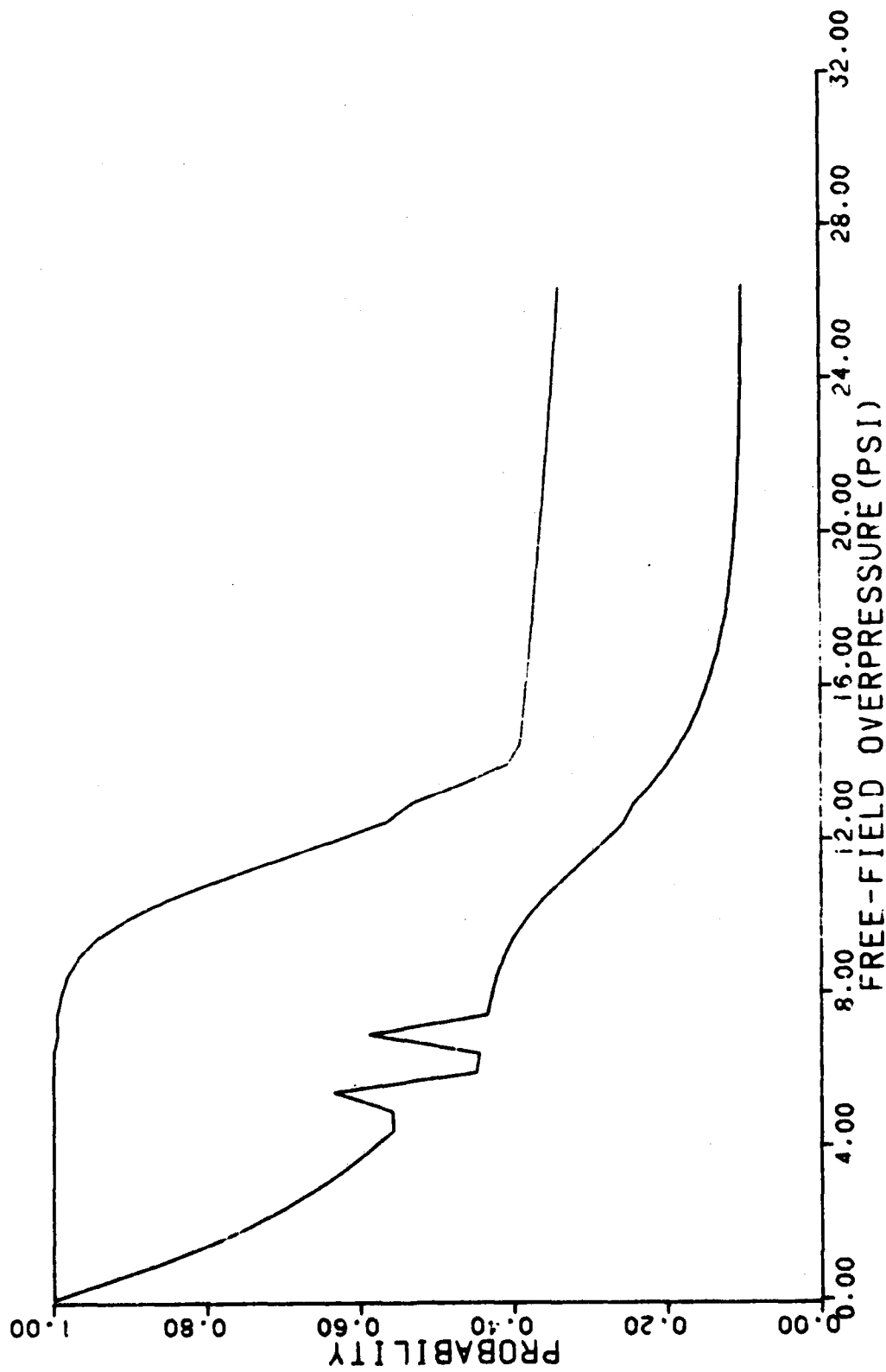


Figure C-102. Probability of people survival (upper and lower bounds) case 11A.

CASE 11B2

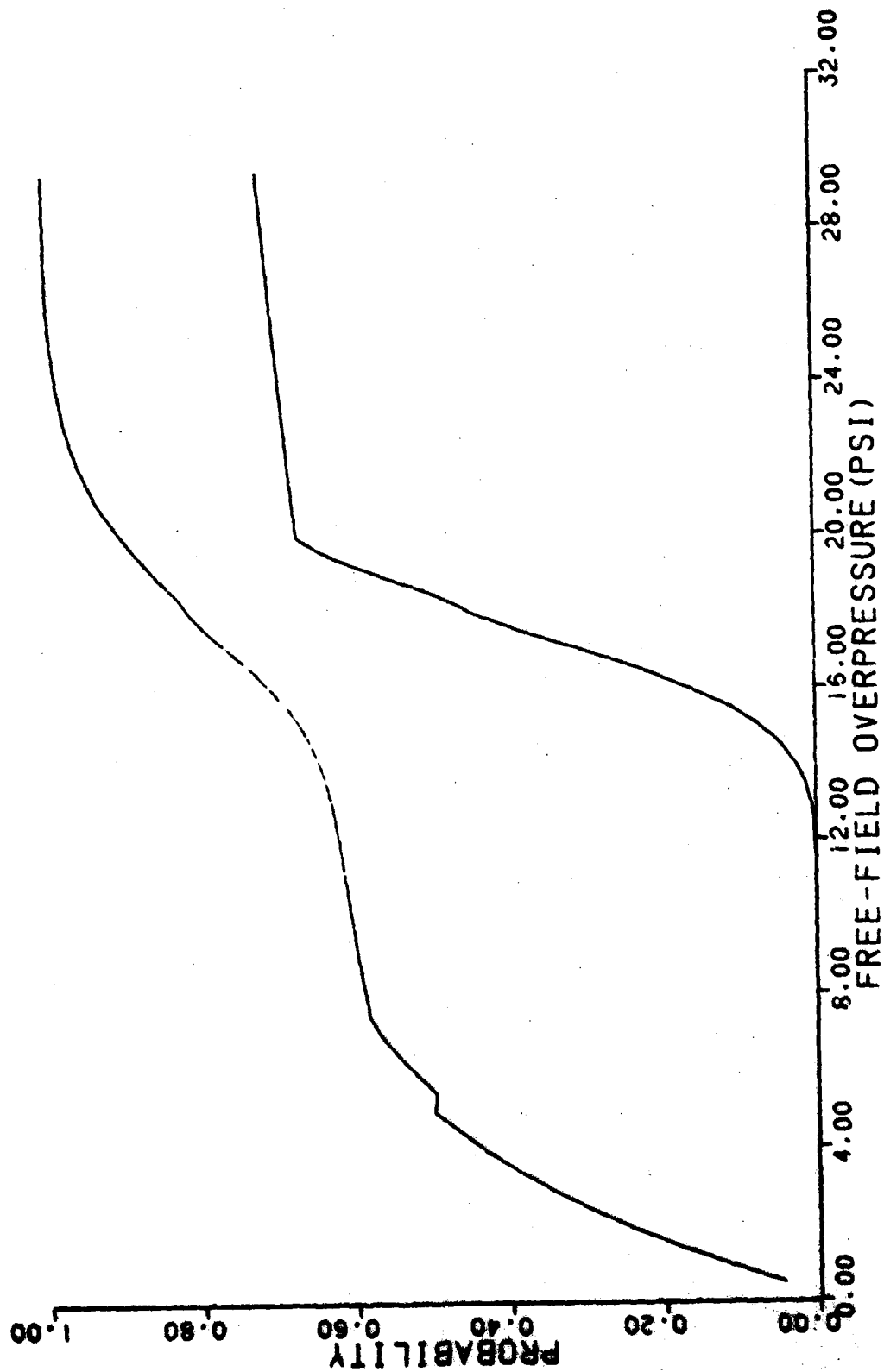


Figure C-103. Probability of slab failure (upper and lower bounds) case 11B.

CASE 11B3

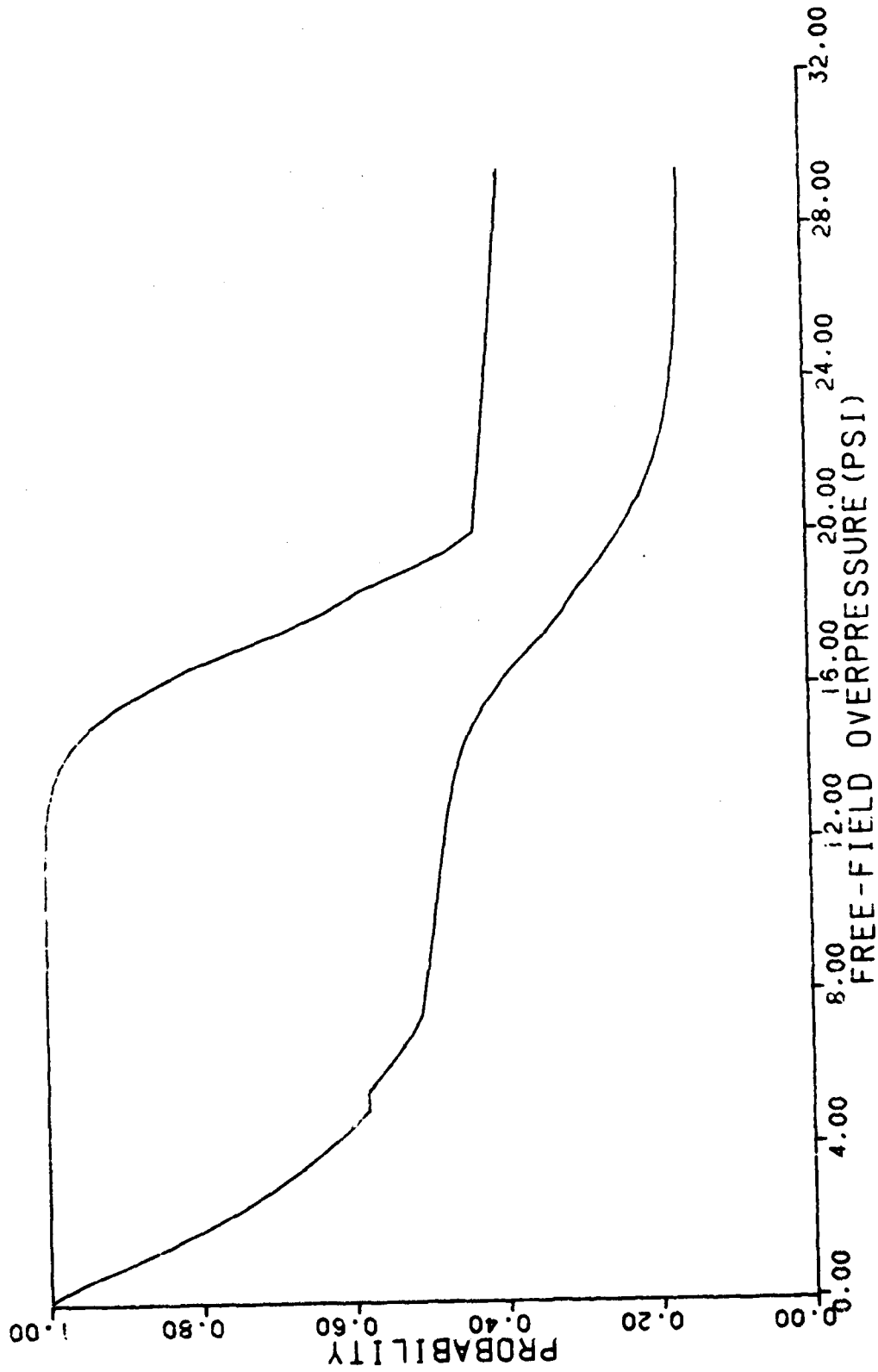


Figure C-164. Probability of people survival (upper and lower bounds) case 11B.

CASE 11C2

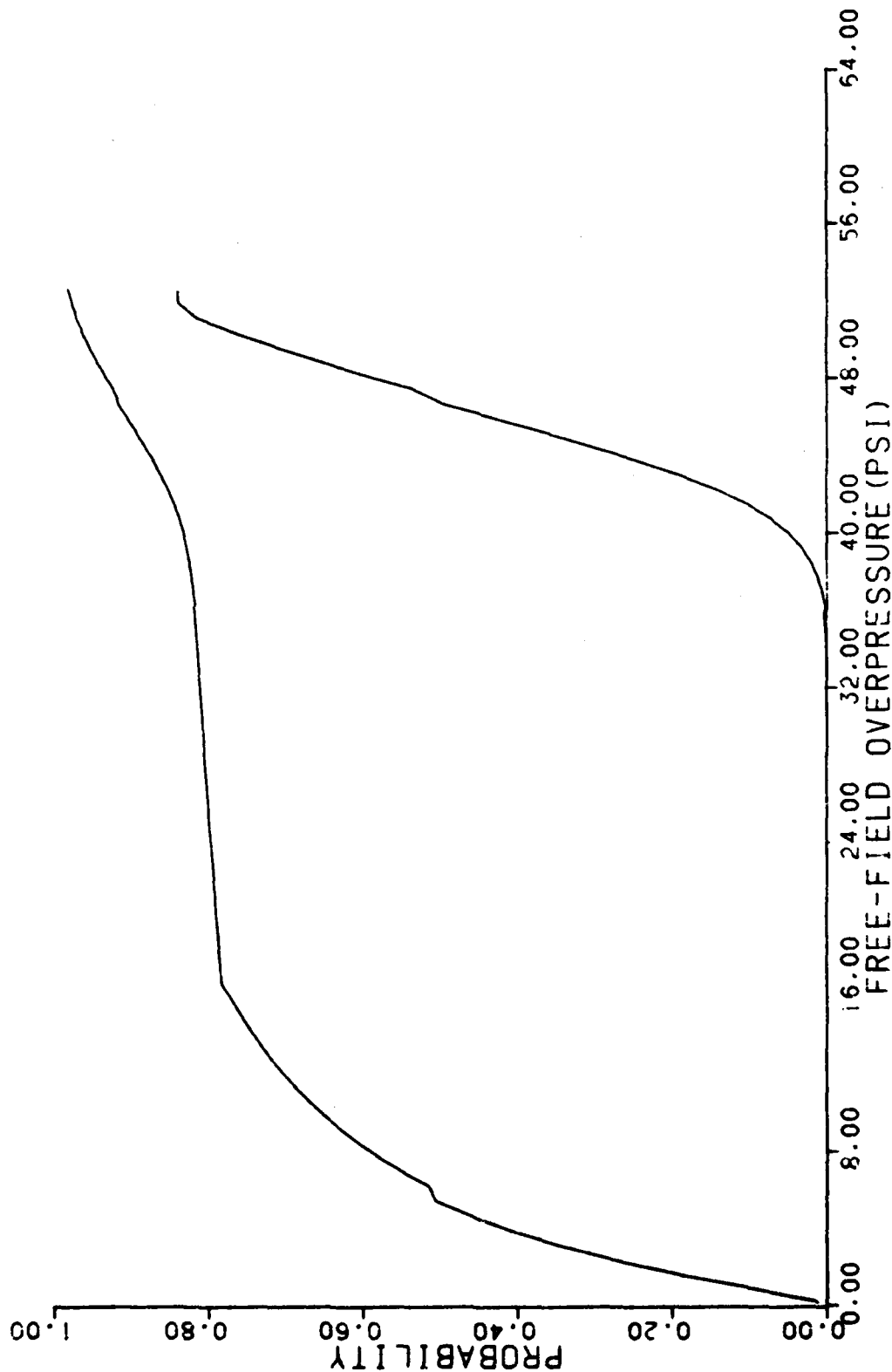


Figure C-105. Probability of slab failure (upper and lower bounds) case 11C.

CASE 11C3

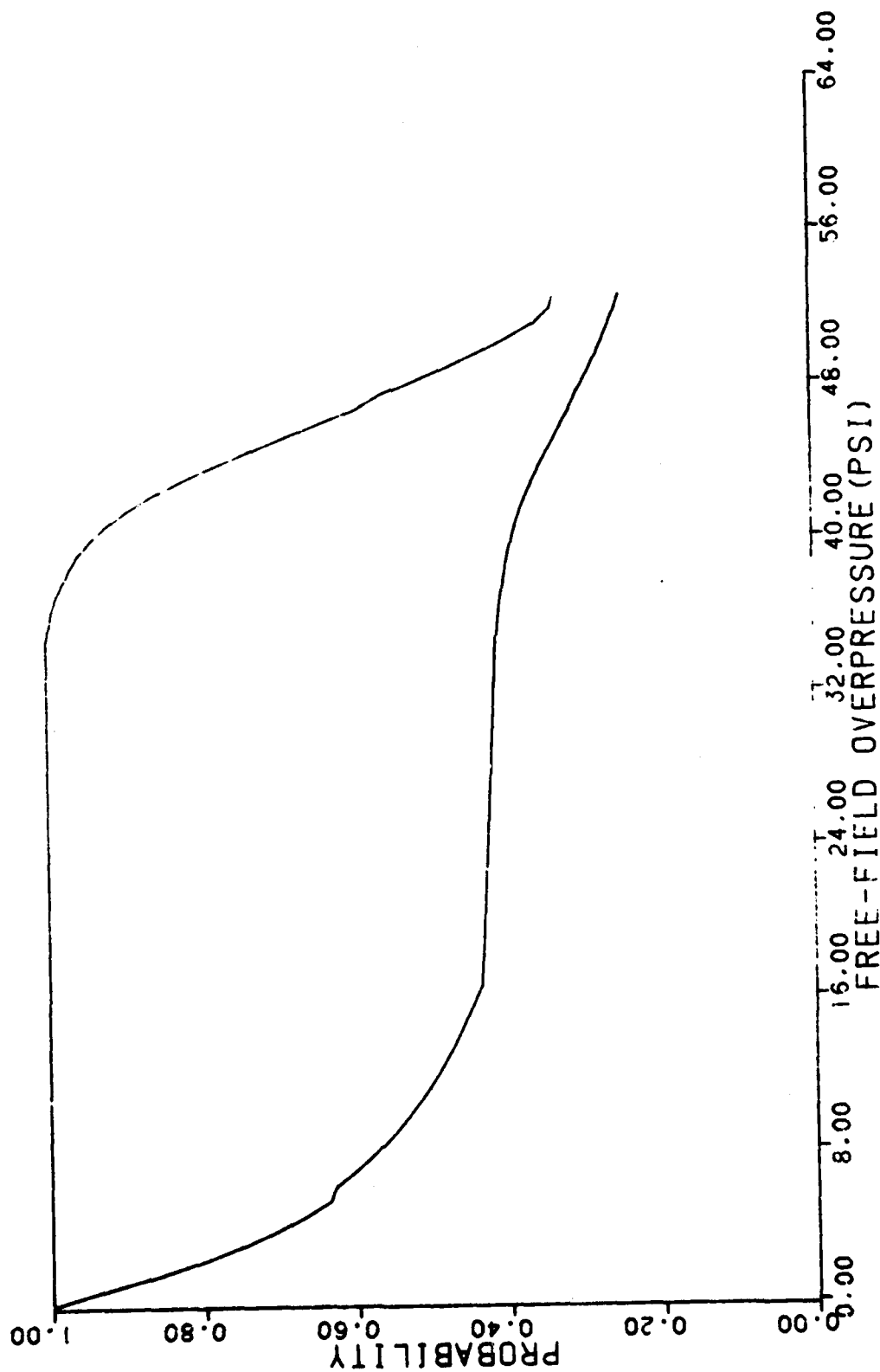


Figure C-106. Probability of people survival (upper and lower bounds) case 11C.

CASE 11D2

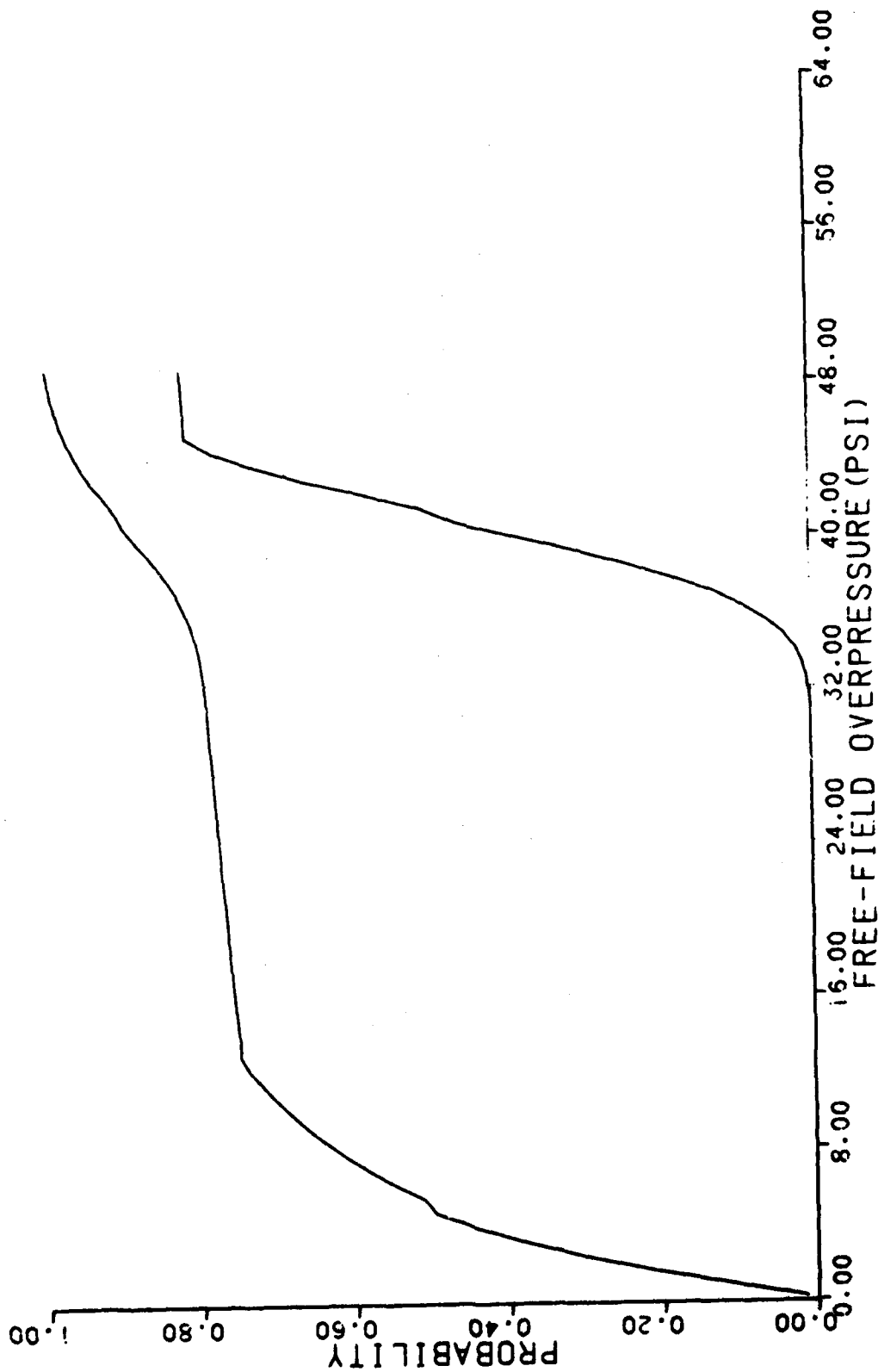


Figure C-107. Probability of slab failure (upper and lower bounds) case 11D.

CASE 11D3

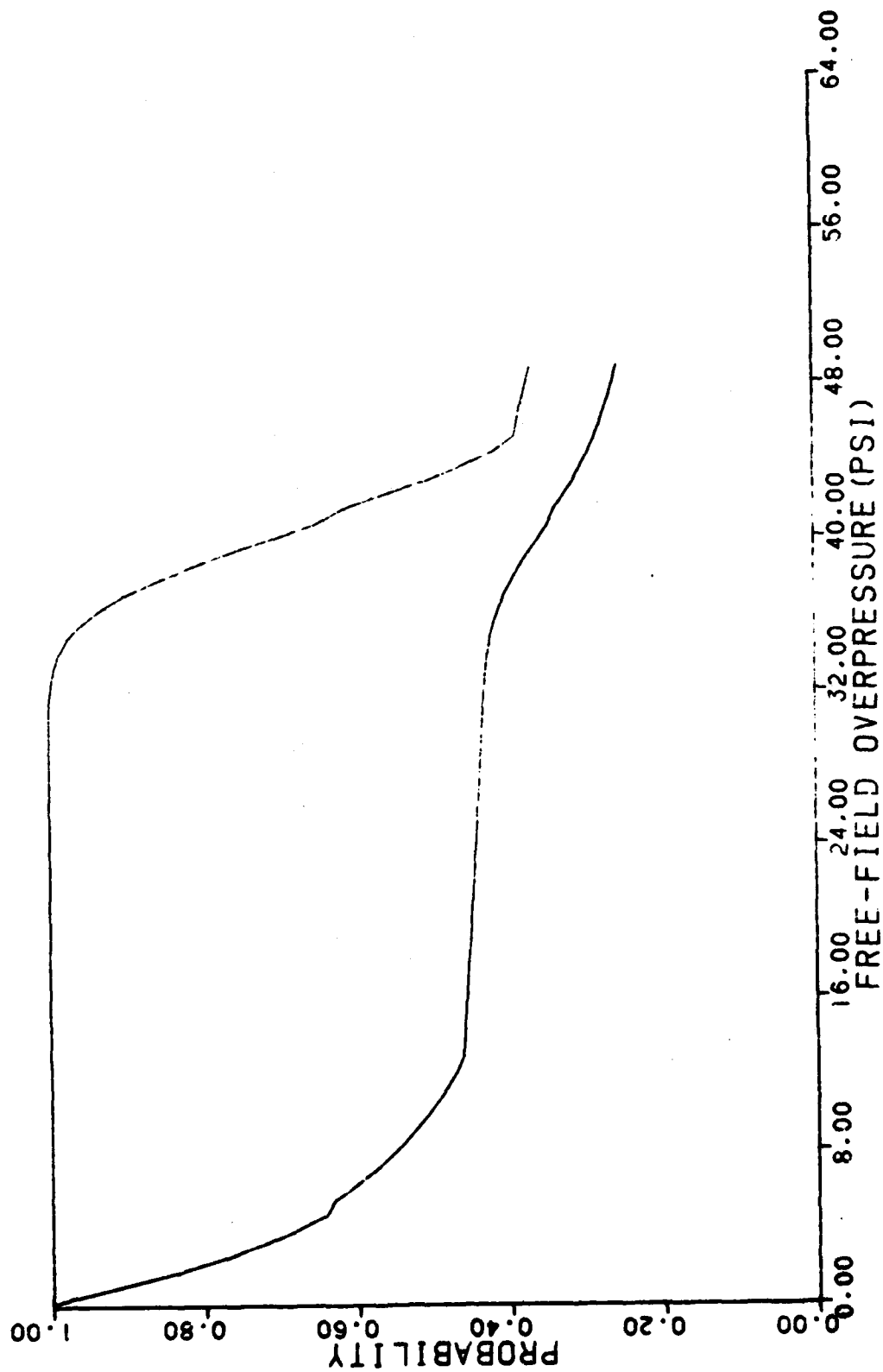


Figure C-108. Probability of people survival (upper and lower bounds) case 11D.

CASE 11E2

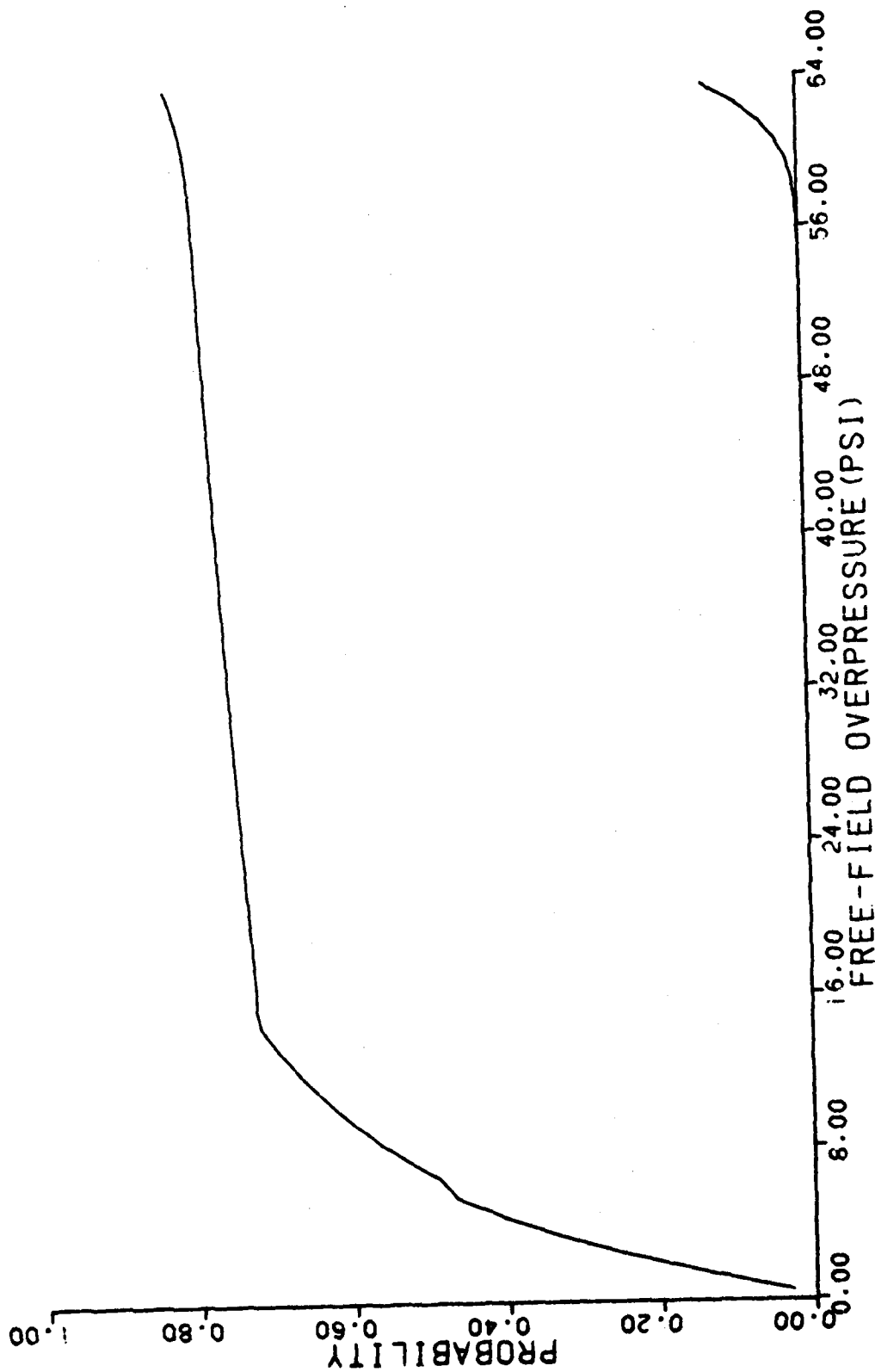


Figure C-109. Probability of slab failure (upper and lower bounds) case 11E.

CASE 11E3

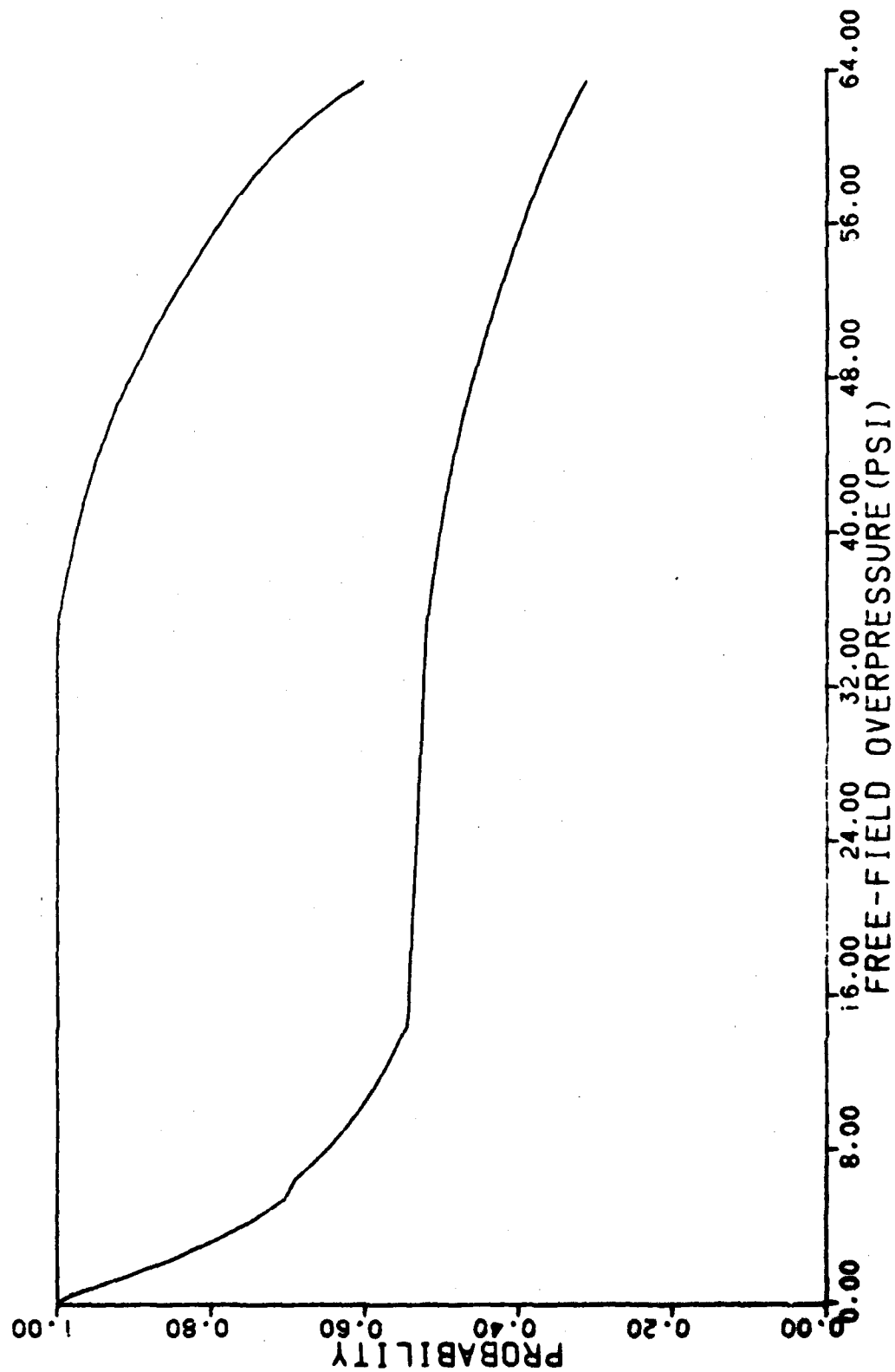


Figure C-110. Probability of people survival (upper and lower bounds) case 11E.

CASE 12A2

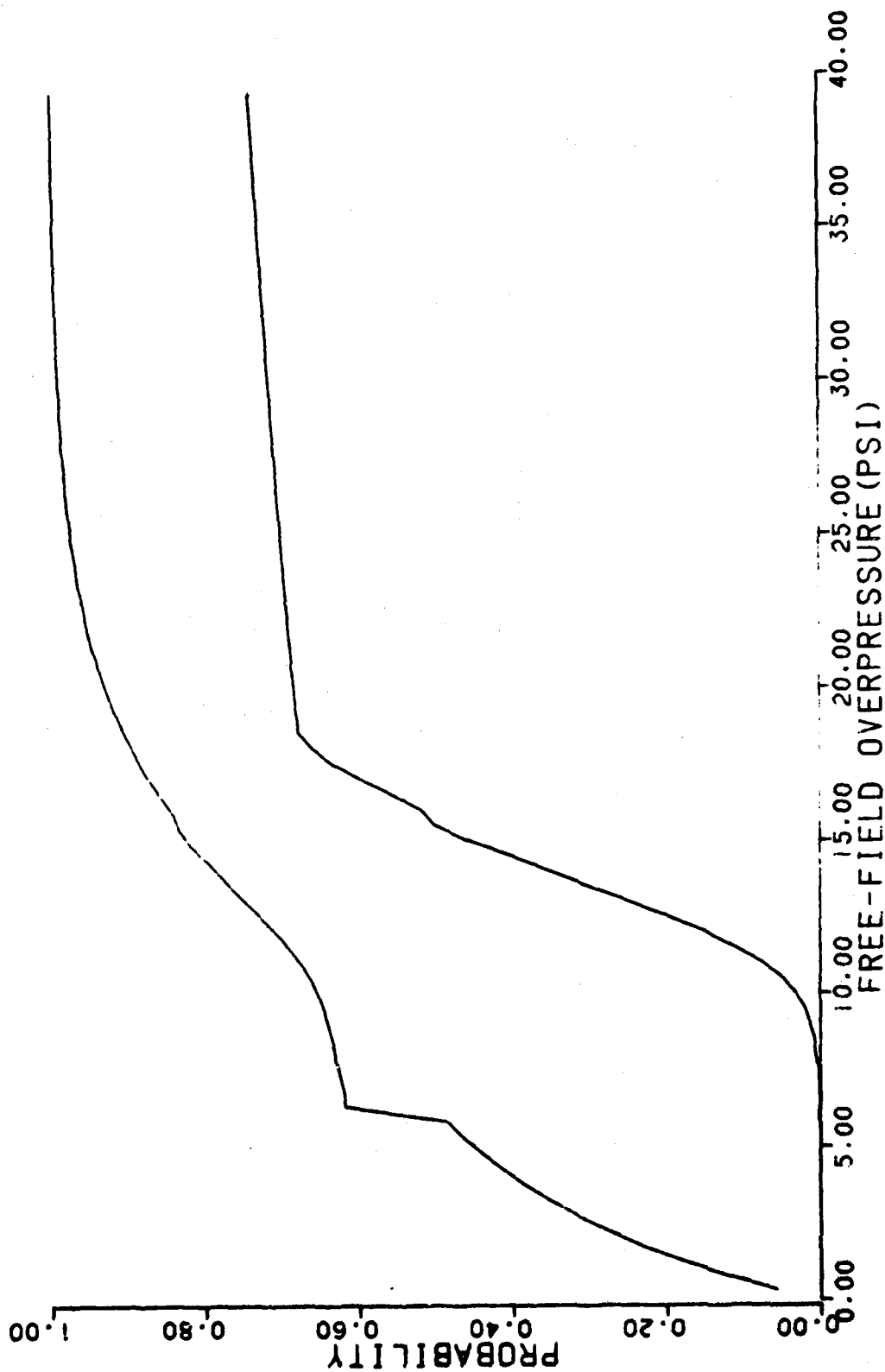


Figure C-111. Probability of slab failure (upper and lower bounds) case 12A.

CASE 12A3

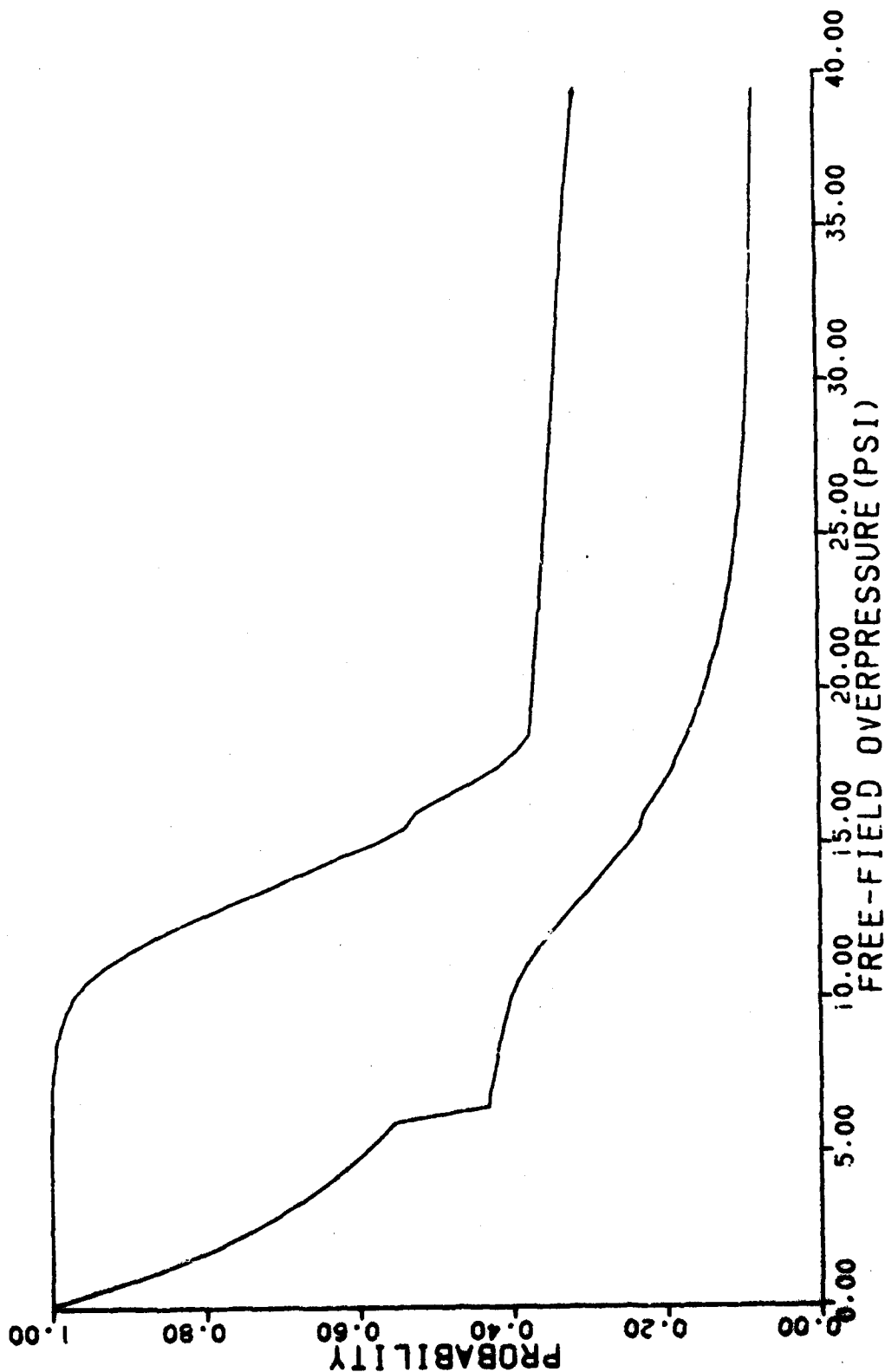


Figure C-112. Probability of people survival (upper and lower bounds) case 12A.

CASE 12B2

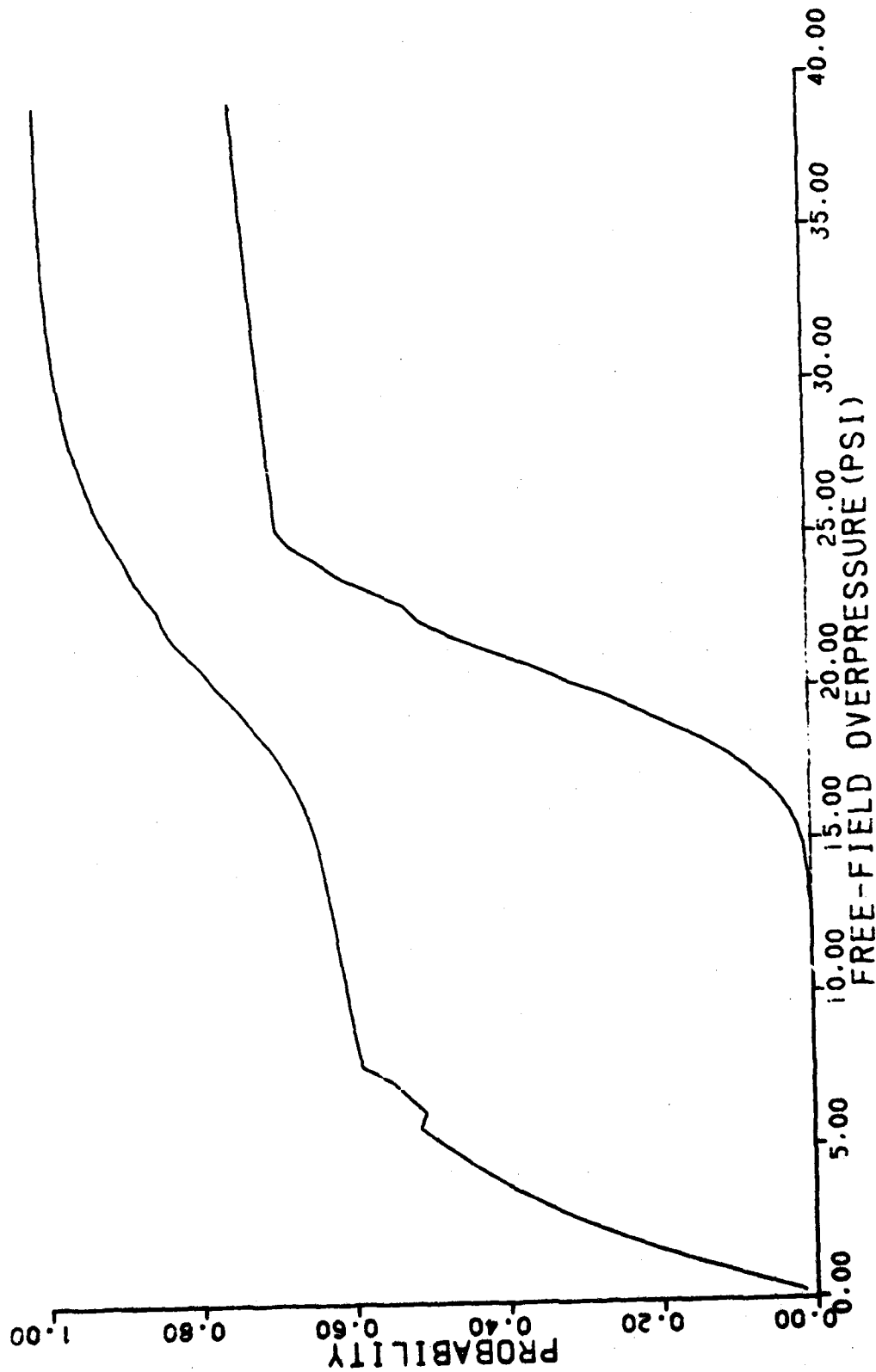


Figure C-113. Probability of slab failure (upper and lower bounds) case 12B.

CASE 12B3

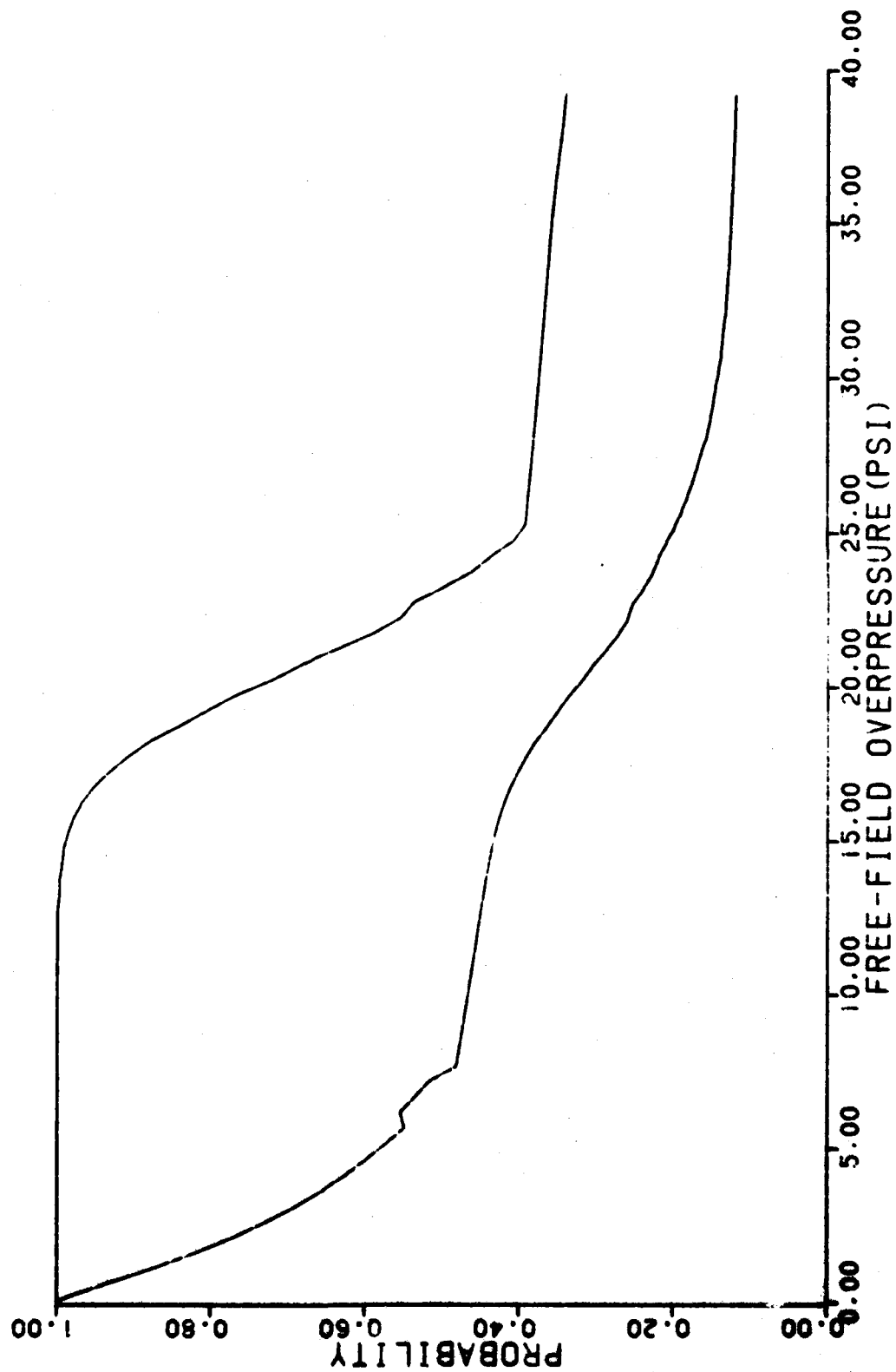


Figure C-114. Probability of people survival (upper and lower bounds) case 12B.

CASE 12C2

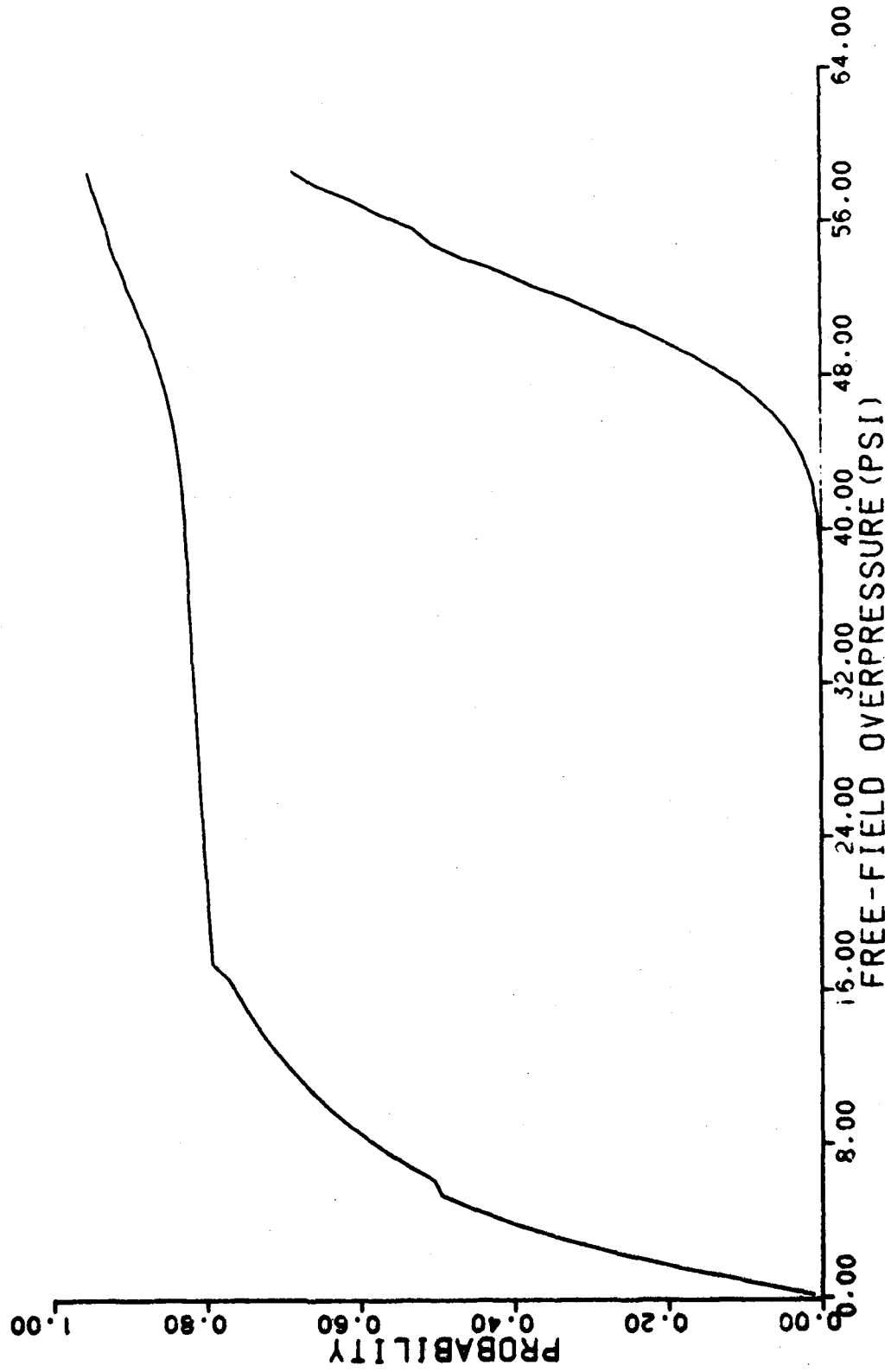


Figure C-115. Probability of slab failure (upper and lower bounds) case 12C.

CASE 12C3

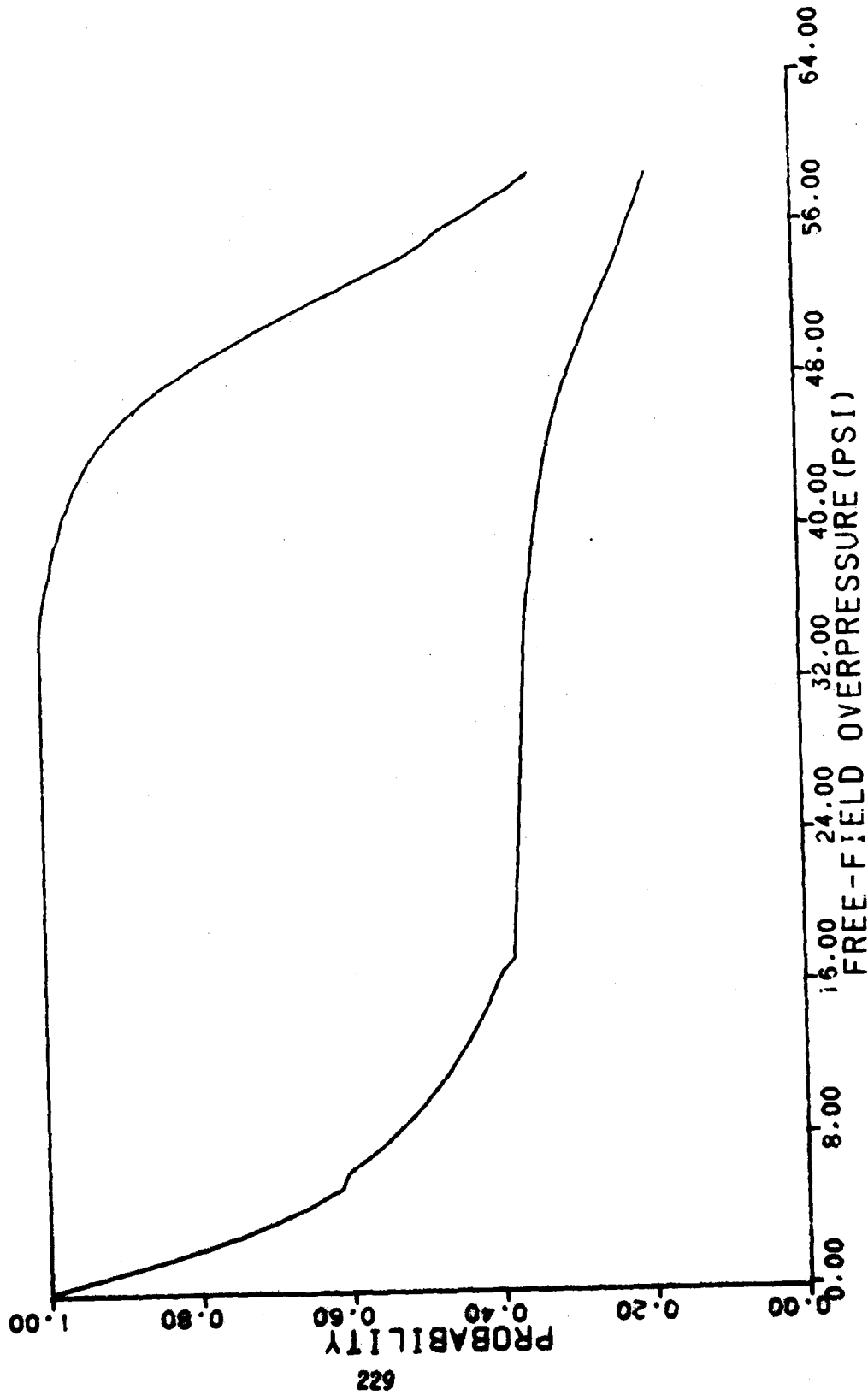


Figure C-116. Probability of people survival (upper and lower bounds) case 12C.

CASE 12D2

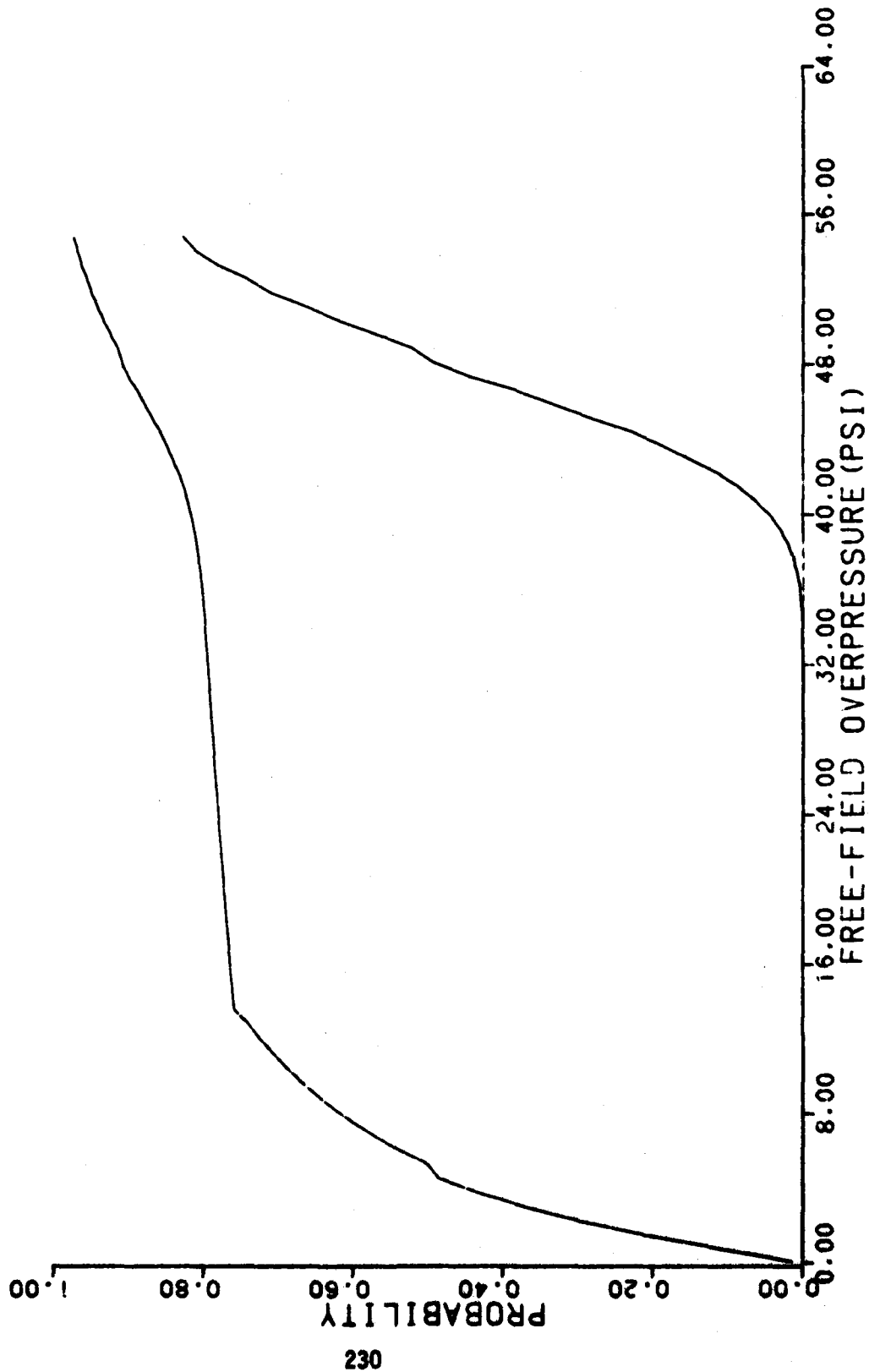


Figure C-117. Probability of slab failure (upper and lower bounds) case 12D.

CASE 12D3

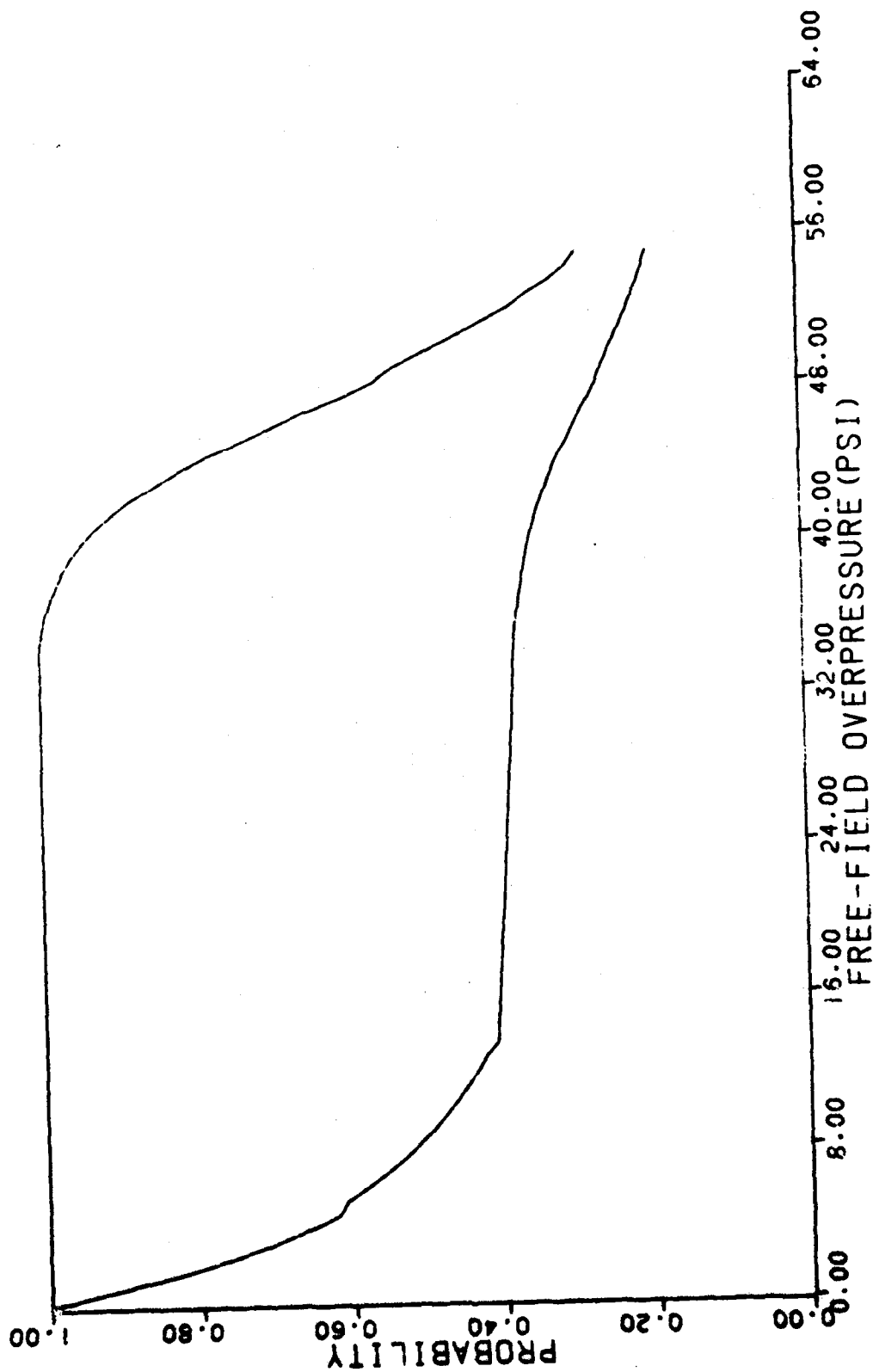


Figure C-118. Probability of people survival (upper and lower bounds) case 12D.

CASE 12E2

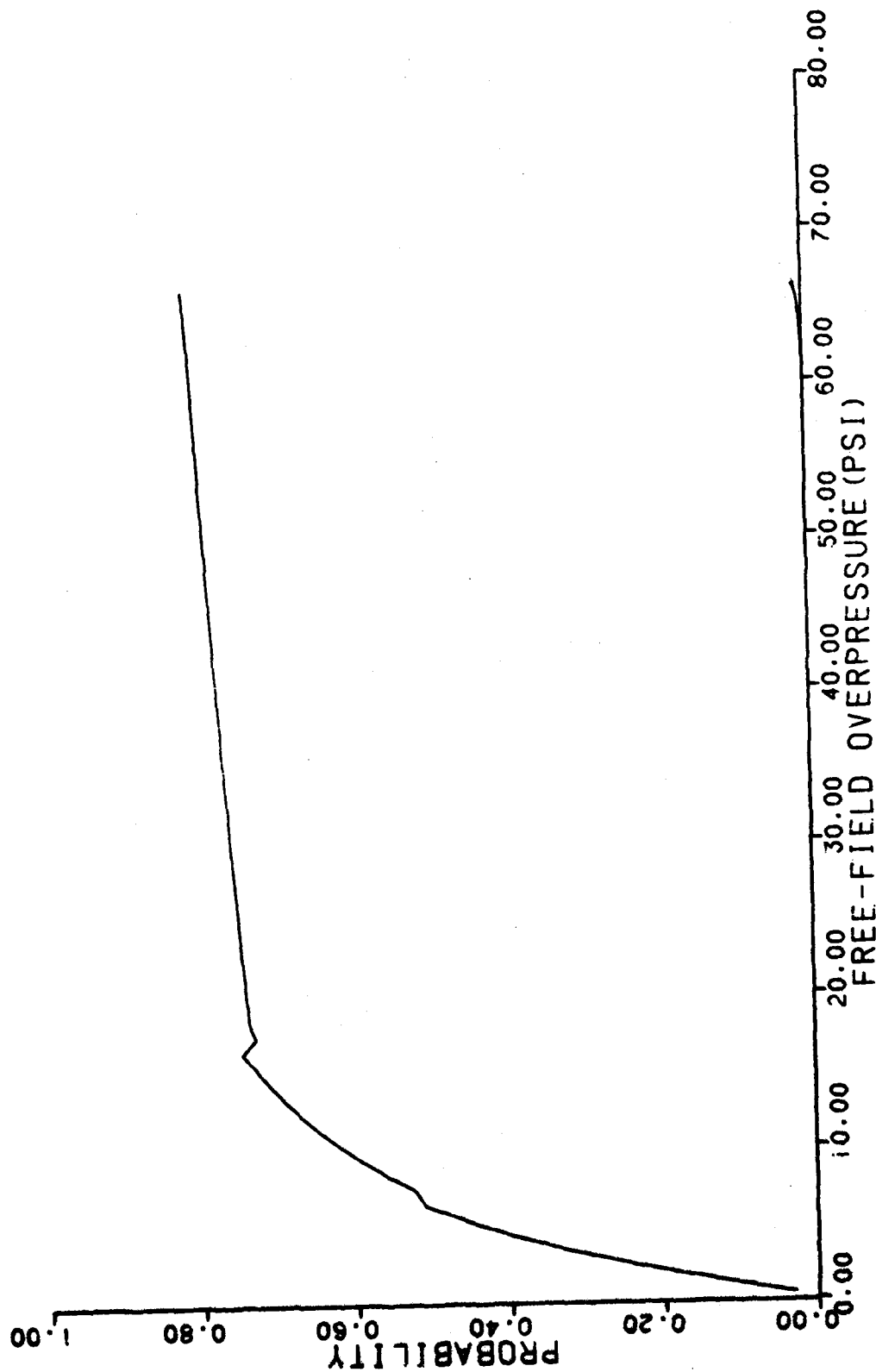


Figure C-119. Probability of slab failure (upper and lower bounds) case 12E.

CASE 12E3

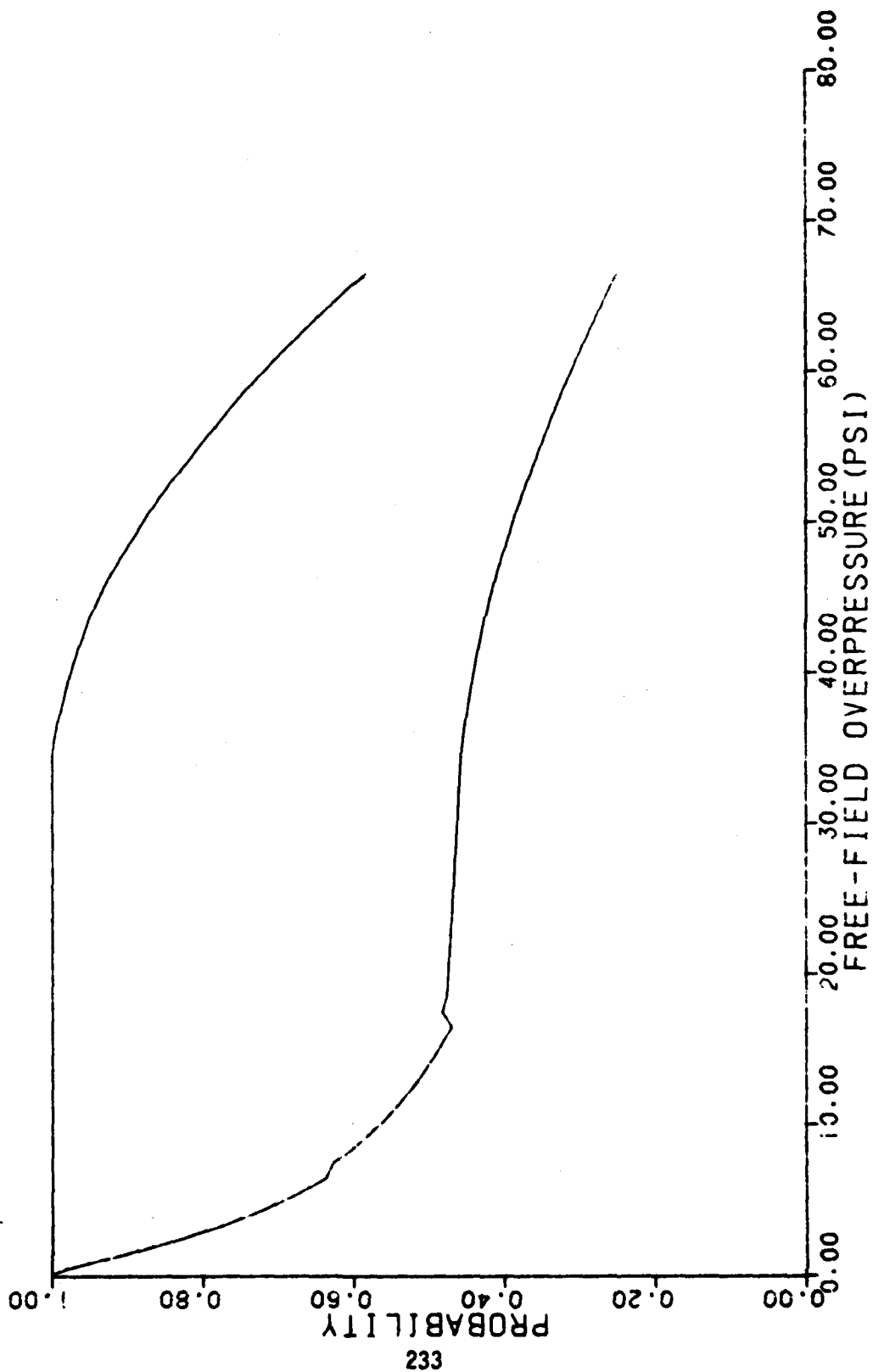


Figure C-120. Probability of people survival (upper and lower bounds) case 12E.

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DAMAGE FUNCTIONS FOR UPGRADED SHELTERS

Final Report

Contract DMR-C-0374

(Unclassified)

IIIT Research Institute

August 1982

ABSTRACT: The probability of survival is predicted of people located in conventional, expeditiously upgraded basements when subjected to the blast effects produced by the detonation of a 1-lb weapon near the ground surface. The categories of potential shelters are considered here, i.e., engineered buildings and single-family residences.

The first category included 12 basements designed for live loads in the range from 50 psf to 250 psf and slab spans from 12 ft to 20 ft. Each of these was analyzed as expeditiously upgraded using four different upgrading schemes. An expedient upgrading scheme involves strengthening the slab over the basement by providing intermediate supports and blocking off all openings into the basement. This resulted in 50 shelters of different strengths which include the conventional, unupgraded slabs as base cases.

The second category included four conventional single-family dwellings with full basements. Each was evaluated when upgraded using a "studwall" upgrading concept. Two of the basements were reevaluated using the "post and beam" upgrading concept. These upgrading concepts are essentially similar and were used as intermediate supports for strengthening the joist floor system.

A probability of survival function was developed for each shelter and each upgrading scheme. The procedure used to accomplish this consists of two parts. The first is a probabilistic structural analysis which determines the probability of failure to the shelter envelope. The second is a probabilistic people survival analysis which considers two casualty producing mechanisms, i.e., debris effects from the collapse of the overhead slab and primary blast. The probability of structural failure is made use of in computing the probability of survival against debris effects.

The report includes a description of the shelters analyzed, a description of the method used in performing the analysis, detailed results, conclusions, and recommendations.

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